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OF

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Montana Bituminous Conference and National Road Oil and Asphalt Congress

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PROCEEDINGS PUBLISHED BY THE STATE OF MONTANA
AND THE MONTANA STATE HIGHWAY COMMISSION

Montana Highway Commission

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LLOYD A. HAGUE, Chairman

LEE M. FORD, Member JOHN M. WHEELER, Member

D. A. McKINNON, State Highway Engineer

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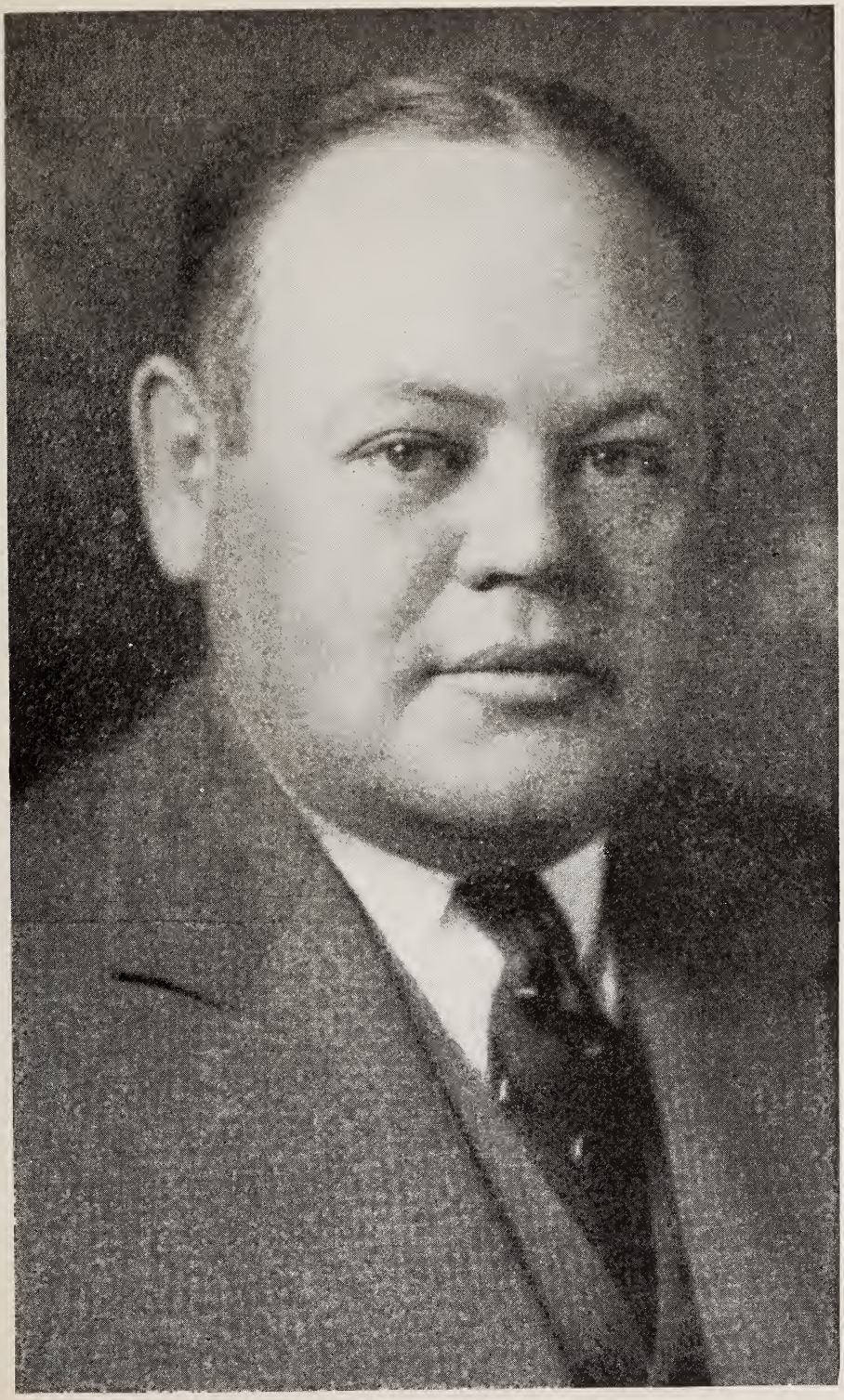
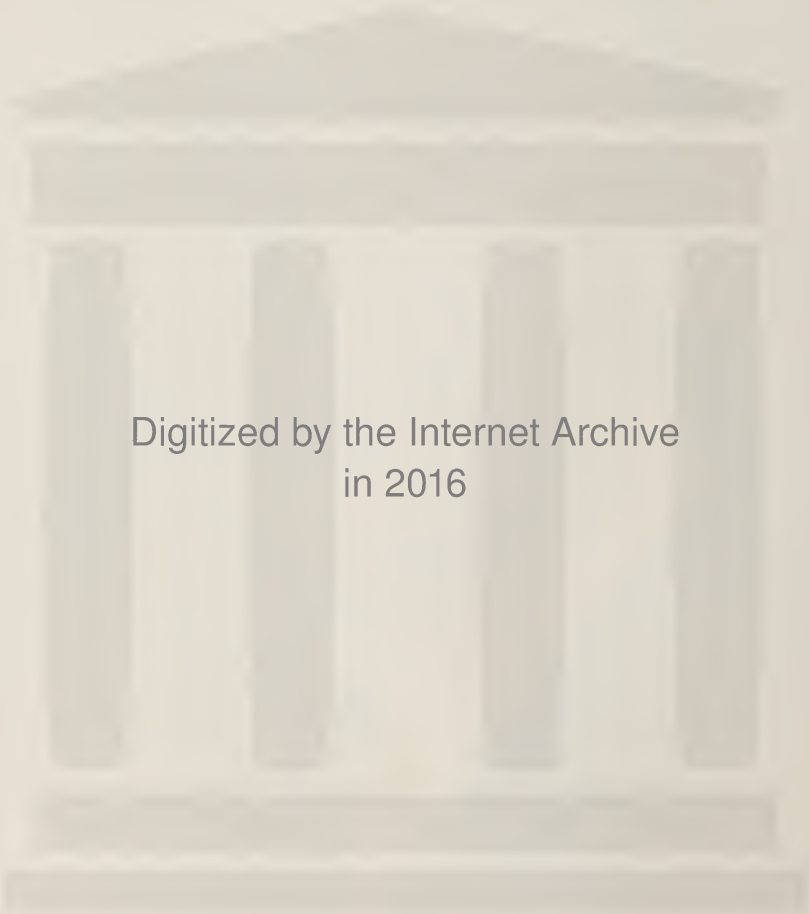


D. A. McKINNON
State Highway Engineer, Montana State Highway Department

General Chairman
1938 Montana National Bituminous Conference

D. L. CHENEY
Bituminous Engineer, Montana State Highway Department

Assistant General Chairman
1938 Montana National Bituminous Conference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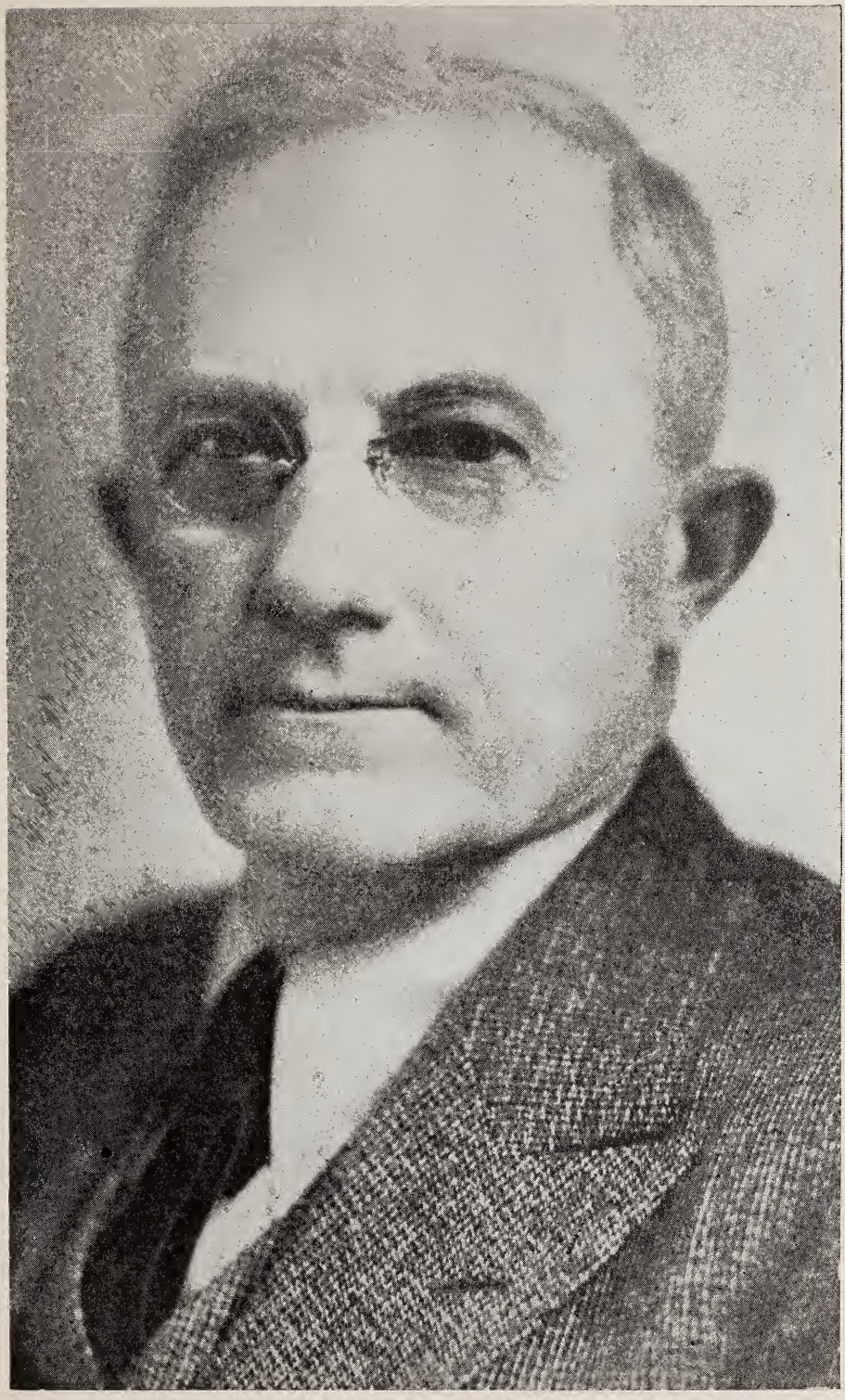


C. M. BOGGS
President Western Petroleum Refiners Association

Vice-Chairman
1938 Montana National Bituminous Conference

OPENING SESSION

Montana National Bituminous
Conference

Edgewater Gulf Hotel

9:30 A. M.

October 10th, 1938

D. A. McKINNON, Montana State Highway Engineer
Montana Highway Department
General Chairman

D. L. CHENEY, Bituminous Engineer
Montana Highway Department
Vice-Chairman

C. M. BOGGS, President
Western Petroleum Refiners Association
Vice-Chairman

Address of Welcome

By

E. D. KENNA, Director
Mississippi Highway Department

Mr. Chairman, Distinguished Visitors, and Fellow Mississippians: I esteem it a privilege and an honor to have the pleasure of standing before you people this morning and delivering to you on behalf of the great Governor of our beloved state words of welcome. The Governor asked me to express to you his very great regret that he was unable to attend, but due to illness it was impossible for him to come down.

These Mississippi boys have my speeches all numbered, from 1 to 12. The shortest, of course is No. 1, and the longest is No. 12. Last night some of them asked me which I intended to use, and I told them No. 12. After last night, however, I have changed my mind, and have decided to make it No. 1.

A few weeks ago I was up in the northern part of our state with our distinguished Commissioner for the Northern district, Mr. Linker, and I made a few speeches on that trip to service clubs. I got over to Columbus and had a telegram from Mr. Harris, Chief Engineer of the Highway Department, in which he stated that complaints were pouring into the office about my speeches, and suggested that I switch from No. 6 to No. 9. I wired Mr. Harris that as a judge of good speeches I though he was a fairly good engineer.

A few years ago, you will recall (all of you who are old enough to recall it) that we had a little war over in Europe. It was my privilege to serve in that war, both on this side and the other. Coming back from France in July, 1919, and going back to my old home county of Pike, all of the boys had in the meantime returned, and the good people of Pike County decided it would be a fine gesture on their part to give a home-coming celebration to all of the boys who had been in the service. And so they prepared a very splendid banquet in McComb, and a large number of people were there.

For some reason I was put on the program and I worked diligently and prepared what I thought was a very beautiful speech of appreciation, and finally my time came on the program and I was introduced by the Toastmaster, and I got up and waded into my speech. Just when I started to reach the point where I thought I was cutting star dust the folks began to get up and drift out, until it got right embarrassing—so much so that the Toastmaster rapped on the desk for order and said, "Just a minute, ladies and gentlemen; just a minute! This man who is now speaking to you went through hell for us, and we ought to be willing to do the same thing for him."

We boys down here in Mississippi have been working hard, catching a little hell here and there, getting ready for this occasion, and I feel that I am entitled to give you folks just a little hell this morning.

We are happy to have you people with us, you visitors from far and near to the beautiful Gulf Coast of our great state. We are proud of the Magnolia State; we are proud of her people. I believe that with all due deference to the citizenship of every other state in this nation, possibly because of the climate, we are perhaps the most hospitable people in this nation.

Secondly, my friends, it is a happy privilege for those of us who happen to be connected with highway building to have as our visitors you distinguished gentlemen who are engaged in that activity and in similar work over this nation. Mississippi is building a highway system. We are glad for you to see as much of it as possible. We are proud of the work that is being done in this state. Our road building was delayed, due principally to financial difficulties, for this is strictly an agricultural state and our finances have not permitted us, until recent years, to really launch a major road building program. But two years ago—nearly three years ago, now—there came into the office of Governor the Honorable Hugh White, a great man and a successful business man, at the same time there came a group of men and women into the legislature who were committed to the task of bringing the State of Mississippi out of the dust and mud.

The legislature and the Governor, in conjunction with our three splendid Highway Commissioners, with a little help that the hired hands could give them, set their hands to the task of bringing Mississippi out of the dust and mud. On May 19, 1936, we held our first letting under that program. From that day until now we have really been working in this state and we are proud of the work that has been done. We have tried to take advantage of the mistakes that other states have made, and in conjunction with Mr. Sneed, the District Engineer of the Bureau of Public Roads, and Mr. Davis, the Senior Federal Highway Engineer in Mississippi, we have been working together in cooperation on our state highway system, and that cooperation we have had from Mr. Davis and Mr. Sneed, together with the information we have been able to get from other states in this nation, has enabled us to do what we are doing, and we are proud of it.

We want you to see it. We want you to go fishing out here in the beautiful Gulf. We want you to see the beautiful scenery, the historic points of interest that cluster around many spots in this state. We want you to see our beautiful women. We honestly feel that we have the prettiest women in the world down here in Mississippi. We want you to see them; we want you to feel at home while you are here.

Now may I say to you again in closing that Mississippi extends to her visitors today a handclasp of welcome, a heartful of joy that you folks are with us, and a hope that your stay in our midst may be one of happiness. It is our sincere hope that you will come back to see us again.

Thank you!

Message to the Montana National Bituminous Conference

From the Association of Asphalt Paving Technologists

Your very competent and affable Chairman, Mr. D. A. McKinnon, honored the Association of Asphalt Paving Technologists by suggesting in a letter that I address this conference. Reluctantly I had to decline this kind invitation because other matters nearer Chicago had prior claim on my time during the week of October 10.

As President of the Association of Asphalt Paving Technologists, it is my pleasant duty to bring you cordial greetings on behalf of our entire membership. Some of you are members of both groups, and it was with this thought in mind that efforts were made to schedule our coming meeting simultaneously with yours. Circumstances over which we had no control prevented such a desirable arrangement. Our meeting will probably be held in Chicago sometime in February. I should like every one here to feel that he has an individual invitation to attend our sessions. We will be more than pleased to have you with us.

I am sure that I voice the opinion of all of our group when I tell you that our Association is heartily in accord with your discussions and efforts to develop the practical side of bituminous paving construction. Converting the results of the study and research of the technologists into practical application is considerably simplified by the cooperation, initiative and resourceful ability of those actually in charge of such construction. Without this active cooperation, a much longer period would be required to make applicable in construction practices the scientific facts developed as the result of such reserach. Thus, the men who represent your group and the technologists of our group have added a multitude of benefits to the traveling public to whom both of us are, of course, indebted for our existence.

There are still innumerable problems confronting us in our efforts to build the greatest highway system known to man. Problems of construction and maintenance, of technology in development and manufacture of materials, of the application of present scientific knowledge to actual usage, of safety—with which we are all concerned—and others too well known to warrant individual mention.

I believe that nearly everyone will agree with the statement that our combined efforts in the past have been of inestimable worth and value. A great many problems under investigation are still to be solved.

We, as technologists, want to continue our researches but must have your help to develop our thoughts and our information into worthwhile

conclusions. I am sure that the same spirit which has motivated men of your group and responsibility in the past will continue, and that we shall all direct our combined energies to enlarging and improving our efforts in that direction.

I thank your Mr. McKinnon for the privilege granted our Association to be identified with the program of the Montana National Bituminous Conference and sincerely regret my inability to be present in person. Based on last year's meeting at Glacier National Park, I know that the various sessions of the Conference will be found most interesting and instructive and all present will materially benefit from the week spent in Mississippi. Thank you.

W. L. HEMPELMANN, President,

Association Asphalt Paving Technologists.

The Contractor's Viewpoint

By

M. W. WATSON

Chairman, Highway Division
Associated General Contractors of America
Topeka, Kansas

Mr. Chairman, Gentlemen of the Bituminous Industry: I received a request a short time back to appear on your program as a representative of the Associated General Contractors of America. Our organization is the national trade association for the contracting industry. It represents not only the highway contractors but the heavy construction, railroad and building contractors as well. The work of our Association is divided into three parts, one of which is the Highway Division, of which I happen to be Chairman this year.

I can speak to two parts of your group from personal experience, having served as State Highway Engineer of the State of Kansas before I became a contractor.

The third group, the manufacturers and salesmen of material, is one with which I have never been associated, and naturally all my contacts with them have been on the side of purchaser.

I was a little nonplussed when I saw that this meeting being held here in Mississippi was a Montana Conference. I couldn't quite understand why Montana would come to Mississippi, but I supposed it was for the same reason that the Kansas Contractors go to Kansas City, Missouri, to have their annual meetings. It is natural for us to go from Kansas to Missouri to have our annual meeting. Kansas has for fifty years been a dry state. But coming from Montana, to a state which has a reputation for being a dry state, to hold a convention, was a little mysterious to me. It did remind me, though, of a situation I found myself in during the early stages of the NRA, when I was in Washington representing Kansas contractors producing aggregates.

It happened that the World Series was on in Washington at that time and together with a distinguished attorney from the State of Montana, who represented the Montana Producers, I attended the last game of the World Series. We had bought our tickets from a very fine gentleman, a fine southern character from North Carolina who had a bunch of tickets, and he was kind enough to sell us these tickets. We were very

comfortably seated and had watched a couple of innings of the ball game when a big bruiser came down the aisle and demanded to know if we had tickets for the seats we occupied. Well, of course, we thought somebody had claimed they had tickets for the seats in which we were sitting, so we immediately threw out our chests and said, "Yes, we have tickets for these seats."

He said, "You come with us. Those tickets haven't been paid for." So we were directed down through the grand stand to the box office, and immediately surrounded by ten or twelve of the largest policemen I have seen in my life, and were informed that at about the time the game started the check for which these tickets had been paid was returned to the office showing that there were no funds in the bank.

They wanted to know where we came from. My friend from Montana said, "I come from Montana." The Captain of Police looked rather queerly at him and said to me, "Where do you come from?"

I said, "I am from Kansas."

He said, "This is certainly a queer situation. Here are two tickets bought on a bad check from North Carolina and the people occupying the seats are from Montana and Kansas. I don't understand it, and I think you ought to go to jail."

I think this is just a kind of mixed-up proposition here, having a Montana meeting in Mississippi, reminds me of our situation in Washington. Of course we finally were able to convince these distinguished minions of the law of our innocence and everyone was happy, and I hope you will have a happy and successful meeting here.

There is a direct relation between the three divisions of the industry which you represent, in which there should be the heartiest cooperation. You men who are in the manufacturing of road oil, asphalts, must market your products through one of two sources. You either must market them direct with the State Highway Departments or with the counties or some other political unit, or you must market them with the contractor who takes a contract from a governmental unit.

There has been quite a trend in the past six years which to the contractors is rather alarming, and that is the trend toward the construction of highways and other public works by what is known as the day labor system. I do not believe that the material and machinery and other interests have considered this problem as seriously as they should. Naturally a man who is selling something is not caring so much whether he sells to one person or another, so long as he sells it and gets his money out of it.

A great many of the companies selling products of the kind represented here lose sight of the trend that has developed, largely due to our relief situation, toward the construction of public works by day labor. We hope that you will give some thought to what it really means should the trend develop beyond its present status. Should the public decide that they want to do all of their public work without having a contractor, would not the next logical step be that they would decide that as well

as doing contract work by the day labor system, "Why don't we go ahead and take over the asphalt industry, take over the sand and gravel industry and the rock industry? And, why stop there? Why not take over all industries."

There are too few business men in the country, seeing their county boards, their township boards and their other governmental units going out and building roads by the day labor political system, who stop to realize that that is only one step toward a socialistic or a communistic form of doing business.

The general public feels that the building of a road is a thing that is mighty easy to do. Every man at some time in his lifetime has ridden on a road, lives next to a road, and feels that he knows all about roads. You who are highway officials realize that day after day people come into your offices and try to tell you more about the roads in the state than you are supposed to know. They think that they can build a road just as well as anybody.

If we go ahead and develop further the day labor system will it not tend toward the point where they will do away with engineering and decide that anybody can build a road: "Let's just go out and built it!"

That situation not only confronts the material men but the engineering profession as well. The contract system of building public works is one that has developed for efficiency. In the first place we must have, in order to let a contract, a definite specification and a complete set of plans. Preliminary investigation must be made which requires the services of an engineer. In order to assure the public that they are going to get a dollar's worth of work for every dollar expended, you have a public "letting." You advertise for bids and you let the work to the lowest responsible bidder. In that manner the taxpayer realizes that he is going to pay a definite fixed sum of money for the project being undertaken.

On the other hand, you most generally start out on the day labor system without any preconceived idea of what you are going to do, without any definite plan, and the public has no definite knowledge of what it is going to cost, and neither does the department which is letting the contract.

The argument is often used that we are in too big a hurry to prepare plans. That argument may stand up in an emergency, where a road washes out, a bridge washes out, or something of that character; but in most cases you have been thinking about this particular project for several years and have had plenty of time to have prepared plans. Some engineers feel that by doing this work by the day labor system you are strengthening yourself with the public. I doubt very much if that will prove to be a fact in future years. Many public bodies have attempted day labor construction for the reason that they secure by this method a more direct contact with the labor on the job and by so doing would have a greater control over their votes at the next election. That is, for a time, true; but in the end any political party or any political organization that wishes to stay in power in any of our units of government must show to the public

that the work which it is doing is economical, that they are spending the taxpayer's dollar in a proper manner, and that he is getting value received for everything that is spent.

In the past few years we have been through a sort of hysteria due to the relief situation, but in the end organizations which are built up on a principal of "We don't care how you spend the money, just so we keep some men working" are going to react to the detriment of those who are responsible for their being in existence. There is going to be a reaction among the thinking people of this country and most of the people in our country do think, if they are allowed to do so without being stirred up by some particularly active organization or by some calamity such as we have had in the way of unemployment. These people are going to realize that they are going to have to go down into their own pockets to pay the bill and are going to wonder if the work done by the agencies doing the work by day labor has been done in such a manner that they have received their money's worth for their expenditure.

It is not my intention to criticize the people who are engaged in necessary relief work, but I do feel that the methods employed are frequently extravagant and that construction work could have been handled in a much better manner if it had been organized on the basis of sound methods which have been successful in the past.

In the early stages of the relief problem I attended a national meeting of the Associated General Contractors of America in which Mr. Harry Hopkins, the head of the relief department, was one of the principal speakers. For several days prior to his appearing on the program the contractors discussed that we were in an emergency and we should offer the services and the organizations of the contractors to the Government, the same as we would offer them in case of war, as this was a war on the depression. It was agreed that we would offer Mr. Hopkins the services of the contracting industry and tell him that we were standing ready and willing to assist him by turning over to him our organizations, our equpiment and our own services on a very nominal and established price basis, so that he might have an efficient organization to handle the construction work in connection with relief.

When this matter was presented to Mr. Hopkins he said very definitely, "We could use contractors, but we won't." There is no logical reason why the construction industry should be the dumping grounds for all of our relief problems or that we should attempt to assimilate the unemployed from all other industries to the exclusion of workers who have been trained and skilled in construction work.

We should all stop talking about needing appropriations for construction work for relief. We need appropriation for construction work in order to carry out needed improvements, and they should handle relief as a social matter. Construction projects should be handled under the contract system by engineers and construction men.

Large corporations, such as the railroads, never attempt to enter into construction projects of any magnitude without letting contracts to men who

are skilled and trained and equipped to do that particular class of work.

Major Gen. Lyttle Browne, who was once head of the U. S. Army Engineers Corps, when asked about the question of day labor, said, "When I was a young man, out in one of the engineering districts doing work, I wanted to do it all by hired labor; I didn't want the contractor to have a look-in; because in my bumptiousness and conceit, I thought I could do it cheaper than any contractor could. And now, even today I admire that spirit on the part of the young men out there in charge of work. I admire that spirit; but I do not admire that judgment, because their field of view is limited. I feel certain that in the long run, the contractors of this country, the great builders of this country, with proper encouragement, can do all the work that we have to do and do it more economically than any government agencies can do it."

At the start of the relief situation the Bureau of Public Roads, as you know, required every highway department to build one job by the day labor system after that particular job had been advertised and bids had been received. The records of the Bureau of Public Roads show very conclusively that the greater majority of these jobs would have been built far more cheaply by the contract system than they were actually built under the day labor system. In fact, some of them ran as high as eighty or ninety per cent above the low bid that was given at the letting, and in one state the Bureau of Public Roads was obliged to withdraw Federal aid from the project because the State Highway Department, in order to keep the cost below the bid, was violating its own specifications. This convinced the Bureau of Public Roads that the contract system was the proper way to do work, and we feel sure that they will continue doing their work along that line.

To leave that subject. Of course, I can not keep from discussing the day labor system among a group of engineers and contractors as it is one of the greatest problems with which we have been confronted in the past few years—this great trend that we have had toward day labor to entirely eliminate from construction the trained contractors with their organizations and their investments in equipment. But there are some other observations I would like to make to you as well.

Your particular type of construction, is one of the leading types of road construction at the present time. The trend in styles of road building seem to rotate from a cheap type to the high-priced type, and then back to a cheaper type of road. In my early experience we were building a great amount of waterbound macadam roads. Prior to the advent of the automobile the waterbound macadam road, the turnpike, was considered a very high type of construction. Engineers tried to find some way of eliminating the raveling and dust hazard, so we began to build the bituminous macadam road.

Then came the cement industry and the brick industry, and with their promotional facilities they showed that it was better to build a higher and more expensive type of road, and these were built during the period when money was more plentiful with people generally.

Then along came the depression, and we have dropped back to the building of a type of road which can be built with less cost than the high-priced brick and concrete roads, and I have observed, in traveling over the country, that a large mileage of our roads are now being built by some kind of bituminous method.

The contractor, naturally, is not as much interested in what type of road is built, as long as he is able to build that type and do so at a profit. But he is interested in seeing that there is a sufficient volume of that business from the contractor's standpoint. The greatest trouble, with your oil or other type of bituminous roads is the shortness of the season during which they can be built. We are obliged to equip ourselves with a lot of equipment at a large investment; yet we are unable, especially in the North and central part of our country, to work this equipment for any great length of time.

A problem is to find what to do with this equipment and with our organizations during the period of the year when we can not work on bituminous construction. I hope that out of this Conference will come a study of some method of handling additional work with the kind of equipment that is used in constructing the oil mat or bituminous type of road. There is a field in which the contractors' organization can be used during the months of the year when they can not engage in bituminous construction. That is in the maintenance of roads by contract.

I believe that the engineers in this gathering should give considerable thought to working out a method whereby considerable maintenance work that is being done by the day labor system could be handled by a contract method. We have in the State of Kansas in the last two years been able to contract with our Highway Department for a sizeable volume of reconstruction of roads which formerly had been done by the day labor method. It has given the contractors a volume of business and allowed them to keep their organizations together which could not have been accomplished had our Highway Department not gone into that phase of the work by contract.

In that respect I feel that our department particularly has been fortunate in securing an engineer who served for a period of years in the organization of a contractor. If every highway engineer of a state, had some experience in the contracting line, it would be better both for the contractors and the organizations of the highway departments.

There is no reason why there should ever be a serious conflict between engineers and contractors. There has been in some instances antagonism between the two. I believe it is getting less and less each year as the engineers and contractors both become more familiar with working with each other. In some instances the professors of engineering in our schools are partially responsible for this situation. They sometimes instruct students in engineering that when they get out into the field of engineering where they have to look after work that is being done on contract, they must watch the contractor, that he is a crook; "You have to watch him."

I have not found that to be the case, and since I have been on the other side of the fence I have been more firmly convinced that con-

tractors as a whole are just as interested in getting a good job and building up a reputation as the engineers are interested in getting a good job. And if you both start out with that in view and the contractors and engineers get together more often, as you do in meetings of this kind, and talk over your problems and study each other's viewpoint, you will develop a cooperative system of working rather than being in conflict with each other on the job. You are both working to the same end. We, as contractors, naturally have to look at one viewpoint in a certain respect, and you as engineers have to look at another viewpoint. We as contractors naturally are more vitally interested in the question of profit than you as engineers in charge of the work. Our continuance in business depends upon our being able to make a profit and build up an organization to go ahead with a larger volume of work.

You as engineers are naturally looking more at the quality of the work. You are trying to see that the public is securing what they are paying for, and that your specifications are not being violated. Those are the main diverging points between the contractor and the engineer. But as a whole, if the two who are on the job, the engineer and the contractor, can sit down and talk the problem over, get better acquainted with each other, most of our difficulties can be worked out. It is not necessary for them to throw rocks at each other all the time on the job.

Neither is it necessary for the contractor to try to slip something over on the engineer when he is not on the job. I do not believe you will find that sentiment prevailing. Our contractors have developed highly trained organizations and in most organizations in our section, the contractor himself either has an engineering training or experience as an engineer, or employs engineers in his organization.

In drawing specifications you should bear in mind what the possibilities are for the contractor to do the work. In setting time limits on a job they should not be set too short. Give the contractor fair consideration; analyze the job as to what the average contractor should do; not what the best contractor in your state or section could do but what the average man should do, and set the time limits accordingly. There will be less argument at the end when he comes in and tells you why he couldn't get the job done.

The manufacturers of bituminous materials should consider the question of demurrage on the use of your tank cars, with a view toward fairer regulation. Road work involving the use of bituminous materials is something that is greatly affected by condition of the weather. In order to keep a steady supply of this material it is necessary that we place our orders well in advance. These cars may be several days enroute, and before we can get them stopped we will have a large accumulation of cars and in the event we are shut down on the road more than the free time allowed by the regulators the railroad comes to us for demurrage, even though we can not work on the road.

We have contended with railroad officials that demurrage on tank cars is the most unfair demurrage they can collect. They do not own the cars and they have no real right, in the estimation of the contractors,

to collect demurrage on these cars. The members of the oil industry should cooperate with the contractors to secure a fairer demurrage regulation. The only way we can escape showing demurrage on cars is to show that there is actual weather interference during the time the cars are detained, but as you know, in this work there will be several days after the weather interference when we can not pour oil or put on asphalt, during the time that the road surface and the materials are getting dry enough so that we can apply oil. I believe that we should ask the cooperation of members of the oil industry in attempting to secure fair demurrage regulations on the shipment of tank cars.

I have nothing more in mind today to talk to you about from the contractor's viewpoint, but to say that the highway contractors are very happy to have the business of building bituminous roads. We hope you will continue the building of your type of road and that we may be privileged, as contractors, to build them for you.

Research Attack On Bituminous Road Problems

By

R. W. CRUM, Director
National Highway Research Board
Washington, D. C.

I take as my text this morning the following excerpt from an editorial review of the recent International Road Congress in Roads and Road Construction, a British magazine:

> "The seekers of the perfect bituminous road have a more difficult job than their colleagues working on (portland cement) concrete. There is only one binding agent in concrete and it behaves according to well established rules which are relatively so simple that even engineers are beginning to understand them. Bituminous binders are legion; they are composed of obscure hydro carbons with sesquipedalian names and their behavior, if not entirely unpredictable is at least variable enough to add a spice of adventure to those who are trying to unravel the mysteries. Apparently to prevent the thing from becoming too easy, moreover, the workers in different laboratories have allowed a state of complete chaos to arise in the methods by which the various materials are tested and identified."

I have nothing new to say about the needed research in asphaltic materials. The needs of the situation are well known to highway technologists. But in view of the lack of progress in attacking the fundamental questions, the only way I can see of furthering the cause is to reiterate on every possible occasion the fact that we are entirely unable to specify the characteristics that these materials should have to assure satisfactory results in road and street surfaces.

All we have are methods for identifying certain characteristics that presumably were had by other materials, which were used some rather distant day in the past when the materials came from a few sources and were used mostly in the form of asphalt cement. We are perhaps not so badly off in this country as the quoted editorial would indicate for a good job has been done toward standardizing identification tests, but they are still just that.

In the United States we are using annually in road work about four million tons of asphaltic products at a cost of thirty-eight million dollars

for the materials alone, to say nothing of the value of the roads in which they are used. Isn't it about time that a concerted effort be made to unscrew the inscrutable behavior of these materials? Even a small percentage of failures runs into big money.

I recognize the vast amount of brilliant work that has been done to work out applications of bituminous materials to the wide variety of physical and economic conditions prevalent on the highways of this country. Indeed, considering our inability to write determinative specifications for the binder materials, progress has been almost unbelievable.

From the scientific standpoint, asphalt suffers under the handicap of being a by-product. But, it is the by-product of an industry that depends to an enormous extent upon highway transportation for the market for its principal products and I am unable to see any logic in the apparent lack of interest of the petroleum industry in the efforts of the engineers and scientists to reach assurance of durability commensurate with costs.

Although your presence at this conference testifies to your faith in the value of bituminous road materials, your presence also testifies to your concern over the fact that the jobs are not always completely successful and that the record of failures is impressive enough to point out a serious situation.

Some time ago the Highway Research Board prepared a prospectus for a research project based on the need for determinative specifications and tests of asphaltic materials in which it was said:

> "There is a great need for a fact finding investigation to show the relations of the characteristics of asphaltic materials to the service records of road surfaces in which they are used.
>
> "The primary objective of such an investigation would be the acquisition of information upon which more rational specifications for materials could be based.
>
> "The specifications in use for asphaltic materials are not satisfactory for the reason that they do not depend upon characteristics of known relation to service behavior of road surfaces. They have always depended upon tests for identification of certain properties or constituents. There are two kinds of identification tests that may be used. One is intended to identify certain properties which asphalts that have given satisfactory service are known to possess; the other is for the purpose of controlling the source of the material or its treatment during production. The first type has a valid place in practice, but as yet our knowledge in this respect is inadequate. The development of more rational specifications for asphaltic materials requires greater knowledge of the relations of identifiable characteristics to service behavior than we now have. The only excuse for the second type of identification tests is the lack of quality requirements that will assure satisfactory results. It is lamentable that our knowledge of asphalt and its use is still in such a state that many engineers feel that they must rely upon such restrictive methods. It is not good engineering to be unable to use the most available material of satisfactory quality and it is not good

business to restrict competition when, by so doing, good materials are excluded.

"More determinative tests that measure directly properties that affect the serviceability of asphaltic road surfaces are entirely lacking, and to devise them will require development of an entirely new field of research. For instance, an accelerated test for the weathering characteristics of various kinds of asphaltic materials and their combinations would be extremely useful. Further research into the exact way in which asphalt performs its function of holding the aggregates together against the forces of nature and the loads imposed by vehicles should indicate tests directly related to this function.

"The fact that asphalt technologists have been struggling with this problem for half a century demonstrates that the solution is far from easy. It is indeed a most complicated and intricate affair, but we are not ready to concede that it is unsolvable.

"An examination of the records is an essential preliminary to any further development of rational specifications or to further research work on unknown or not understood factors. Study of the causes of failure as well as reasons for success should be illuminating. It will probably be found that in very many instances usable information concerning the materials is lacking. However, if enough cases are examined it is probable that an adequate number with significant data will be found."

The support necessary for such a project has never been found, but the Committee on Characteristics of Asphalts of the Highway Research Board, of which Mr. E. F. Kelley is Chairman, explored this situation briefly by means of a questionnaire, with very illuminating results.*

The questionnaire was distributed to 189 individuals, of whom 91 replied.

It was apparent from the replies that, "failures or unsatisfactory results in asphaltic highway construction attributable to the use of asphaltic material of poor quality are sufficiently prevalent to merit serious consideration."

The report goes on to summarize as follows:

"The occurrence of failures or unsatisfactory results from this cause is not localized but is found in practically all sections of the country."

"Trouble with the use of cracked asphaltic products appears to be more prevalent than with uncracked products."

"Among the asphalt cements the harder grades appear to be the chief source of such unsatisfactory results as reported."

"Trouble with the liquid asphaltic products is not limited to any particular types or grades but appears to be more prevalent with cracked than with uncracked materials."

"Cracking of the asphalt construction or treatment, and drying out with ravelling or dusting seem to be the principal types of failure attributed to poor quality of the asphaltic products."

*Report of Committee on Characteristics of Asphalts, Proceedings, Highway Research Board, Vol. 17, p. 328.

"Hardening and increased brittleness of the asphalt were the most frequently observed changes in the asphalt producing unfavorable results."

Consideration of these observed results leads to the pertinent question —why? Which means research.

The Committee suggested an approach through the "development or standardization of some rapid laboratory method of determining the relative resistance of asphaltic products to hardening and other changes." It further recommended correlation of such a test with service behavior of asphalts and the selection of limiting values for the test for use in specifications.

Now—what can be done? I believe two major efforts are indicated.

(1) I am still convinced of the need for a thorough fact finding survey of national scope, for it is not probable that all pertinent factors were disclosed by the cursory questionnaire investigation I have quoted.

(2) A correlated laboratory attack based upon the revelations of the fact finding survey is needed to discover ways to measure the abilities of the bituminous materials used in mixtures for road surfacing to coat the particles of mineral aggregate, to adhere to them and bind them firmly together and to resist disintegration. For instance, we need to know why some identification tests will provide for the use of satisfactory materials although by the same test other materials that would be satisfactory are excluded. We need to extend the work being done on correlation of characteristics as measured by ordinary laboratory methods with the properties of bituminous paving mixtures. We need to study the sources of materials to determine how the characteristics of crude oils influence the quality of the finished product. The effects of refinery processes should also be investigated. Little can be foretold about the characteristics of bituminous materials that influence resistance to weathering from present test methods. We know that heating, mixing and placing asphaltic mixtures alters their characteristics, but how can these effects be measured? And then beyond the study of the characteristics of the bituminous materials themselves lies the vast field of their combination with other materials ranging from common soil to elaborately graded aggregates. Mr. E. F. Kelley discussed these factors more thoroughly in his paper on "Needed Research on Asphaltic Road Material at the 1935 meeting of the Highway Research Board.*

When means can be found by which the characteristics of asphalts that affect its ability to coat aggregate particles, to bind them firmly together, to stick to them permanently and to resist the disintegrating influences of weather, can be measured, two most important benefits will be immediately apparent. We will be able to write quality specifications for materials to meet particular service conditions and we will have something to go on in developing technique for the proper use of the most available materials. The latter point deserves emphasis: it is engineering at its best when proper methods can be devised for the utilization of the most economically available material. Any one can write a specification

*Proceedings, Highway Research Board, Vol. 15, p. 261.

that will require the use of special material brought from a distance at great cost, but it takes knowledge to do the other thing successfully. Surely the possession of such knowledge would profit both producers and consumers alike.

Research Organization

None of these things can be done without the sincere cooperation of both the producers and users of these materials. The road builders know most about use conditions, but service can be vitally affected by source of raw material and production processes, and so their knowledge must be combined with that of the refiners if we are ever to reach the goal. To make progress beyond the extremely slow advance of the past 50 years will require coordinated effort.

There are numerous agencies now active in research work that will probably be glad to fit their efforts into the general picture and no subsidy should be needed for this, the larger part of the job. But some means should be found to organize a general agency that can conduct the necessary fact finding surveys, plan and correlate the specific research projects and publicize findings and recommendations. This agency must be subsidized for I do not know of any organization in this field which is competent to represent all interests impartially on a national scale that is equipped with funds for such a project.

I am not here to promote the Highway Research Board as an agency for this purpose, although it is a type of work for which the organization was set up. There are others that could do the job. However, in order to clarify the matter with respect to how the Board may function in such a situation, I should like to take a few minutes to describe our organization and how it would function in operating such a project.

The purpose of the Highway Research Board is to provide a national clearing house for highway research activities and information.

In its practical workings the functions of the Board have been: to provide a forum for the discussion and publication of the results obtained by individual research workers; to organize committees of experts to plan and suggest research work and to study and correlate results; to publish and disseminate research information; to provide a research information service; and to carry on fact finding investigations when possible.

The Board is organized as a project of the Division of Engineering and Industrial Research of the National Research Council. The membership consists of the representatives of 35 technical and commercial associations and organizations of national scope.

Much of the technical work is done by committees of specialists and recognized authorities which are organized under seven departments: Administration and Finance, Highway Transportation Economics, Design, Materials and Construction, Maintenance, Traffic, and Soils Investigations. The members of these Departments are outstanding men in their fields. Special committees are appointed from time to time to carry on specific projects.

Regular contacts are maintained with all State highway departments, with the engineering colleges and with many city engineering departments, through regularly appointed contact men.

Although the Board is not set up to be in itself a working research agency, its organization as a part of the National Research Council makes it possible to furnish auspices under which fact finding surveys of national scope, scientific in character and impartial in their findings can be carried on. Numerous special projects have been organized and completed with the cooperation and financial assistance of other interested parties. The detail work has usually been done by a special investigator employed for that purpose by the Board, under the supervision of a committee of the best informed men available.

The Board is not an endowed institution and it has no general funds for the conduct of research projects. Each one of these must be individually financed.

Through years of experience a procedure for the conduct of special investigations has been evolved that brings the best thought in the country to bear upon a problem and produces well considered results.

For the first step a number of men through the country, representing all interests and well qualified to have worth while thoughts upon the problem are invited to attend a conference. At this meeting all phases of the situation are thoroughly discussed and a record of the discussions made. This record is then transcribed and edited and sent to the men who were present for corrections and further comment. A committee of workable size is then appointed to draw up an analysis of the problem and outline procedure for the investigational work based upon the concensus developed at the conference.

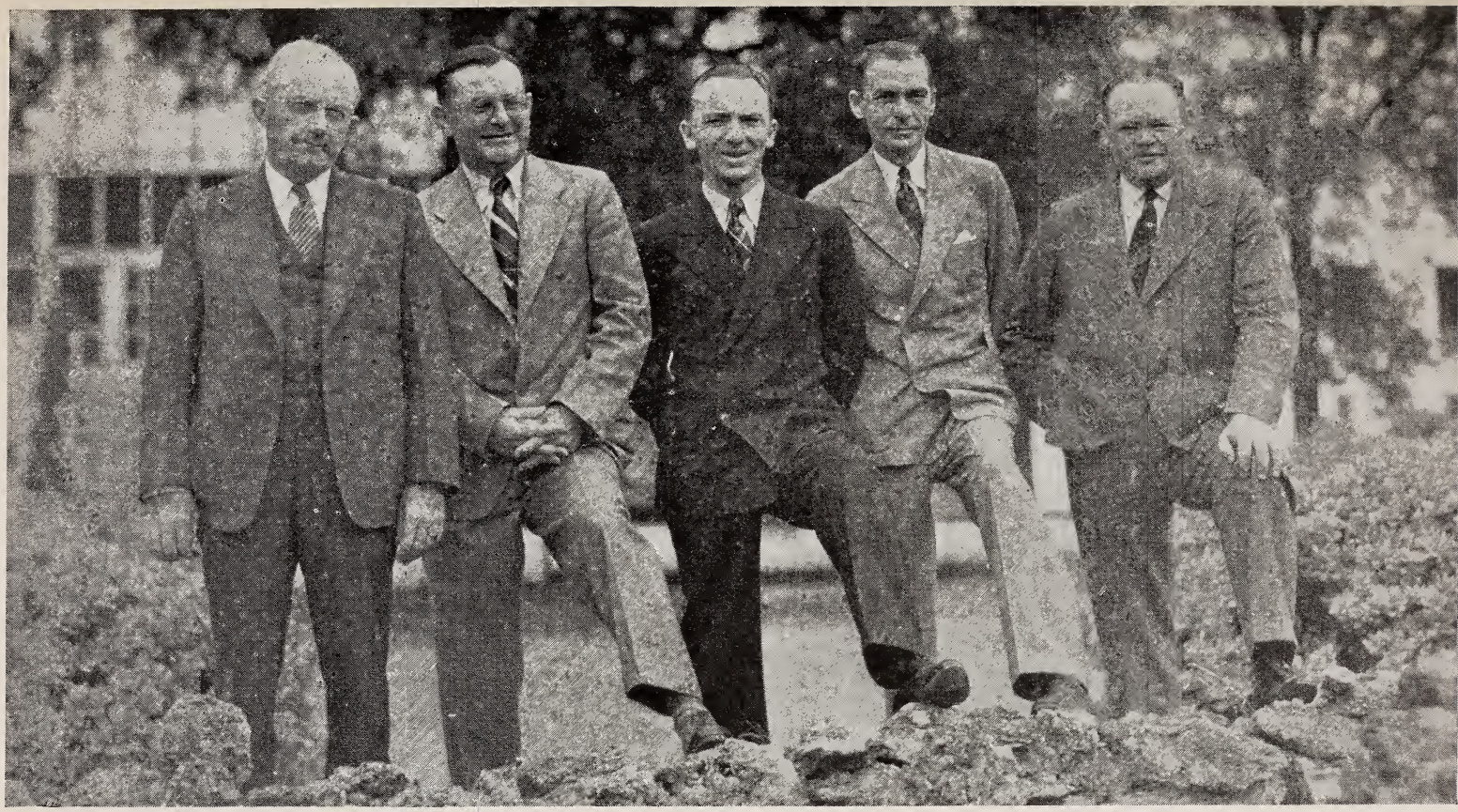
If arrangements have been made to carry on, an advisory committee is then appointed for the duration of the job, a special investigator and the necessary staff is employed and the work started in accordance with the agreed upon procedure. Although all legitimate interests should be represented upon this advisory committee, the guiding principle is to secure the help of the men throughout the country best qualified by training, experience and achievement to bring impartially constructive thought to bear upon the problems as they arise.

Men of the type we want for an advisory committee are busy with important affairs and we must not impose too much upon their public spirit, so it is necessary to staff the project with real research men who do not need too much detailed direction; but the advice of the committee is invaluable in dealing with the larger developments as they occur.

Of course, all reports are submitted to the advisory committee for fine-tooth scrutiny so that when the results are made public we may feel sure, although the final and complete answers may not be reached (they seldom are), that they are technically and scientifically sound.

I have been talking about an ambitious program. Needless to say, the Highway Research Board would be glad to have a part in such a one: but large or small I assure you our business is to try to be helpful in any way possible.

Yes, — asphalts, oils and tars are immensely important road building materials with impressive records of service behind them, but—we have scarcely started to learn all that we should know to use them to the best advantage.



(Left to Right): C. M. Boggs, President Western Petroleum Refiners Assn., Arkansas City, Arkansas, Vice-Chairman, Executive Committee; G. A. Draper, Research Engineer Mississippi Highway Department, Jackson, Mississippi, Chairman, Entertainment Committee; D. L. Cheney, Bituminous Engineer, Montana Highway Department, Helena, Montana, Vice-Chairman, Executive Committee; G. E. Everett, Public Relations Director, Western Petroleum Refiners Assn., Tulsa, Oklahoma, Program Committee; D. A. McKinnon, State Highway Engineer, Montana Highway Department, Helena, Montana, General Chairman, Executive Committee.

SECTION I

Progress in Research as Related to Construction and Maintenance of Bituminous Surfaces

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Progress in Research as Related to the Construction and Maintenance of Bituminous Surfaces

By

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The very fine address on Research presented this morning by Mr. Crum of the Highway Research Board, was an excellent prelude to this afternoons' program. Those of you who heard it, can not fail to be impressed with both the importance of, and the necessity for research in modern highway engineering developments.

In connection with the review I am about to present, it should be emphasized that this is merely a summary of the many excellent papers prepared for the Conference by the members of the Committee on Research. Any credit for the material appearing in this brief survey of "Progress in Research as Related to the Construction and Maintenance of Bituminous Surfaces," is therefore due to these Committee members, who have managed somehow to find time during the busy construction season to

prepare the many interesting papers you will have the opportunity of studying in the printed proceedings later on. It is not possible in this short review to do justice to any one of these well written papers.

The topics upon which papers have been prepared by members of the Research Committee can be divided into four major divisions:

(1) Soils and Soil Stabilization.
(2) Bituminous Mixtures.
(3) Bituminous Materials.
(4) Research Applied to Maintenance.

In this review these four divisions will be dealt with in the order given.

SOILS AND SOIL STABILIZATION

The papers contributing to progress in research on soils and soil stabilization will be considered under the sub-headings of:

(a) Soil Mechanics.
(b) Soil Classification.
(c) Soil Stabilization.

(a) Soil Mechanics.

Professor Housel of the University of Michigan and the Michigan State Highway Department, has prepared a paper on "Developments in Soil Mechanics as Related to the Design and Construction of Bituminous Surfaces."

Considering a bituminous surface as a slab resting on a soil foundation, Professor Housel reviews those problems upon which investigation has been and is being concentrated, in order to obtain immediate beneficial results.

He first considers those problems concerning the subgrade soil or the foundation for the surface.

Because of the deleterious effects of water on bearing capacity and other soil properties, there have been developed, largely through the research of the U. S. Bureau of Public Roads, the plastic and liquid limit tests for evaluating water holding and binding capacity, and the various criteria for evaluating volumetric changes in the soil with changes in moisture content.

Because of the detrimental effects of certain soil freezing phenomena, some interesting research has been carried out in an endeavor to explain these, leading to the constant frost line theory of Taber and the fluctuating frost line theory of Benkelman and Olmstead. Both sets of investigations indicate that soils of silt texture should be removed from the subgrade to the lower limit of the frost line. Taber to the contrary, Benkelman and Olmstead have demonstrated conditions under which the coarse textured sands and gravels may cause frost heaving, and indicate the possible necessity for lowering the water table below the frost line even in these soils.

Accumulation of capillary water in a clay subgrade soil immediately underlying a bituminous surface may lead to its failure because of an increase in soil volume and a decrease in bearing capacity.

In discussing the bearing capacity of soils, with which the science of soil mechanics is chiefly concerned, it is necessary to divide soils into two groups, granular and cohesive, because the factors of soil resistance are different for each group.

There are two distinct schools of thought concerning the bearing capacity of cohesive soils.

One school, sponsored by Professor Housel himself, holds that bearing capacity is controlled by the shearing resistance of the soil, and when failure occurs, it is due to plastic flow of the soil because its yield value is exceeded by the applied load. This conception recognizes that there is some consolidation of a soil for any load, but emphasizes that conditions of equilibrium may occur without continuous, progressive, settlement.

The other school of thought, sponsored by Terzaghi, believes that bearing capacity is controlled by consolidation alone, and excludes the possibility of failure by plastic deformation. They question the possibility of a definite shearing resistance for plastic soils, and point to the present unsatisfactory status of shear tests in upholding their views.

While there is no universally accepted procedure for measuring the shearing resistance of soil materials, some methods have been developed and used in actual practice. Sponsored by the A.S.T.M., some twenty laboratories are now collaborating in an endeavor to standardize these shear tests. Professor Housel developed the penetration method for evaluating the shearing resistance and bearing capacity of cohesive soils some years ago. This method for determining the bearing capacity of these soils has given very gratifying results in the foundation design for a number of engineering structures.

For soils consisting of granular materials, there is again a diversity of opinion as to the factors of soil resistance controlling bearing capacity. For many years the bearing capacity of granular materials has been considered a function of internal friction. However, as a result of research carried out at the University of Michigan on the stability of granular materials, Professor Housel has concluded that the bearing capacity of granular materials depends not on internal friction, but on the arching action or mutual support of adjacent particles in a granular mass.

The two tests used to evaluate the stability of granular materials are the stabilometer test and a shear test.

The stabilometer, as adapted in Michigan, measures the relationship between an applied load and the lateral pressure supporting a sample, when it is tested to failure.

The ability of a bituminous surface to distribute a vertical load depends upon the shearing resistance of the mat. Test procedures for measuring the shearing resistance of granular materials or bitumionus mixtures have been developed by a number of investigators. The A.S.T.M. are at present endeavoring to standardize a test for measuring the shearing resistance of granular as well as cohesive materials.

Several recent developments in testing technique have not received the attention they merit. Vokac has demonstrated that the effect of both the rate of loading and the dimensions of the specimen must be evaluated in making compression tests of bituminous mixtures. While this requires the preparation of a large number of identical specimens, the introduction of testing apparatus equipped with automatic control of rate of loading, and automatic recording devices, considerably reduces the laborious nature of the task of making these compression tests.

Professor Housel's paper on "The Design of Flexible Surfaces" is too well known to require more than a reference to it here. This theory of design has recently been subjected to trial when the Michigan State Highway Department conducted an extensive series of load tests on a flexible bituminous surface over both clay and sand subgrades. While the analysis of these tests is not yet complete, there appears to be substantial agreement between the theoretical design formulae and the data taken in those field tests.

In concluding his paper, Professor Housel points out that most state highway departments and other engineering organizations have already equipped themselves with both facilities and personnel for putting into practice pertinent developments in soil mechanics research, and that rapid progress in the further development and utilization of the principles of soil mechanics by highway engineers may be confidently expected.

Because it presents a viewpoint directly opposed to that of Professor Housel, a paragraph in Professor Crandell's paper should be included here. Speaking of the value of the shear test on soils Professor Crandell writes, "It has been apparent that the original investigators have set great store on shear as the answer to the problem of stabilization. Many pages are filled with formulae for figuring shear. Yet the interesting fact is that all too frequently the mathematically derived answer does not agree with observed results as to the behavior of the soil in field tests. What is wrong? The engineer has been treating soil as though it were a structural member of a bridge, and—it isn't."

In concluding this brief review of developments in soil mechanics, the fine discussion of the variables to be considered in applying the principles of soil mechanics to the design of flexible surfaces, given in a paper presented by Nevitt at last December's meeting of the Assocation of Asphalt Paving Technologists, should be mentioned.

(b) **Soil Classification.**

Highway engineers in general, are so well acquainted with the very fine work of the U. S. Bureau of Public Roads in their endeavor to suitably classify soils, that it may be a distinct surprise to some that any other system of soil classification could have any value in highway engineering.

During the time when the Bureau of Public Roads system of soil classification was being developed, Highway engineers in the State of Michigan were endeavoring to utilize a system for classifying soils which was being even then almost universally used by soil scientists, the pedological system. Anyone who takes the time to thoroughly study the present

well-developed status of the work of the soil engineers in that State, cannot fail to be impressed by the great utility of this latter system of classification.

Mr. O. L. Stokstad, Engineer of Soils for the Michigan State Highway Department, describes this system and its use, and compares it critically with other system of soil classification in his paper, "A Critical Evaluation of Systems of Soil Classification as Applied to Highway Engineering."

Stokstad defines the main purpose of soil classification to be the providing of soil information through the organization of data without the necessity for continuous sampling and testing. He states that the important need for classification no longer exists insofar as a particular problem is concerned, if its solution is already provided by laboratory test results. For this reason the usefulness of the Bureau of Public Roads system of classifying soils into eight groups on the basis of laboratory tests is definitely limited.

The published reports on soil investigations for highway engineering have been largely concerned with such individual and usually unassociated problems as frost heaving, drainage, bearing capacity, fill construction and soil stabilization, together with a discussion of the test procedures involved. Far from sufficient emphasis has been placed upon becoming acquainted with the properties of soils as they exist in the field and are encountered during highway construction operations.

Stockstad believes that test results are not sufficiently associated with individual soils in the field, an association which is necessary if an endless repetition of sampling and testing is to be avoided. In this connection, the system of soil classification being developed in the Argentine Republic is sited as being of particular interest, because of the combination of the pedological scheme of soil classification together with testing procedures designed to indicate the performance of various soils in highway subgrades.

The systems of soil classification discussed in Stokstad's paper are:

(1) Classification by texture as illustrated by Enos' triaxial chart.

(2) Bureau of Public Roads classification into eight groups on the basis of tests designed to reflect performance.

(3) Pedological system of soil classification which identifies soils by separating them into (1) Zonal groups (2) Series (3) types and (4) phases. This is the system used by the U. S. Bureau of Chemistry and Soils and by soil scientists throughout the world.

The Eno classification which is based on soil texture alone, while valuable as a means for differentiating textures, has only limited usefulness because other soil properties such as drainage, permeability, uniformity, etc., may be of equal or even more importance in many cases.

The system of classification developed by the Bureau of Public Roads through the fine work of Hogentogler and co-workers is essentially a laboratory system of classification. The use of the Bureau's eight groups

for classifying soils in the field approach to soil engineering problems does not lend itself to the recording of essential and detailed field information without a large amount of field work and the preparation of voluminous field notes. The soil tests developed by the Bureau are more valuable for furnishing an attack on special problems, as for example the utility of plasticity index in the design of stabilized gravel mixtures.

Highway engineers in general are either not acquainted, or only imperfectly so, with the pedological system of soil classification. Therefore in reviewing the usefulness of this system for classifying soils as given by Stokstad, its utility may be more easily grasped for the purpose of the discussion period to follow, if the following questions concerning it are answered:

(1) What is it?
(2) How is it used?
(3) What are its advantages?

The pedological system of soil classification is essentially a field classification, in which soils are classified on the basis of the characteristics of the soil profile. The soil profile consists of the upper few feet of the vertical cross-section of the soil in any area, which for purposes of study and identification, is divided into three layers, or horizons, any one or more of which may be missing in individual cases. These three horizons from the top down have been labelled the A horizon, which includes the soil layer leached by rains and other surface waters percolating through the soil, the B horizon, wherein the salts and colloidal matter leached from the A horizon are deposited, and the C horizon, which consists of the parent soil material. The thickness of these horizons, when present, may vary from a fraction of an inch to several feet.

All characteristics of the soil profile are catalogued, which might shed light on the influence of the soil on practical problems in any field where soil may be a factor.

For the purpose of identifying any given soil, the soils of a country are divided first into zones or great soil groups, e. g. podsol, chernozem, laterite, and prairie. A soil zone takes in all of the soils in a given area whose profiles have some particularly well developed characteristic distinquishing them from the profiles of soils found in adjacent soil zones. The soils found in each zone are divided into soil series having similar profiles, drainage, topography and parent material. These soil series are labelled by giving them a geographic name taken from the locality where they were first recognized. The names of typical Michigan soil series are Miami, Onaway, Ontonagon, etc. The surface layer of soil in each of these series may vary as to textural class, and may be clay, sandy loam, etc. The combination of the series name with the textural class gives the soil type, which is the unit of soil classification. Thus each soil type is thoroughly indentified in this scheme of classification merely by naming the type, as for example Miami Loam, Miami clay loam, Ontoagan silt, etc. Also, by simply identifying an area on a soil map by means of the name of the soil type, implies that the soil profile in any portion that area possesses certain definite and known characteristics.

Stokstad has briefly outlined the manner in which the pedological system can be used by highway engineers.

From the brief description just given, it should be apparent that there may be a considerable number of soil types in even a small area, and the characteristics which differentiate each type must be familiar to the soils engineer in that district. This means that the soils engineer may have to be thoroughly acquainted with a large number of soil profiles. While this might seem complicated, Stokstad states that it is simple enough that a highway engineer may become quite proficient at soil mapping in less than three months time.

For an individual project, soil knowledge is first used in connection with the location and type of road to be built. If county soil maps are available for the area, much can be learned from them as to the problems which may be encountered on various alternate locations. In connection with secondary highway programs, the usefulness of the soil for agriculture may determine the location in order to encourage or discourage agricultural utilization of the soil in a given area.

When the location of the project has been determined, a complete soil map of the project is made by the soils engineer in the field. The soil is identified and mapped for several hundred feet on each side of the proposed right of way. This survey is made merely by the indentification of the soil profile in the field through borings or by the examination of cuts already in existence. The boundary of each soil type encountered is determined.

When the soil map of the location is complete, it is turned over to the design division, and it is here that a comprehensive soil survey is of greatest value. To assist the design division of the Michigan State Highway Department, a chart has been prepared listing the characteristics and properties of each soil type. This chart, an example of which Stokstad includes with his paper, answers many of the questions regarding the soil, in which the designing engineer is interested.

When the plans are finished and construction begins, the work of the soils engineer consists of examining soil materials in the grade in order that soils of frost heaving textures may be identified and removed, and that a uniformly textured subgrade is prepared. Drainage conditions of the subgrade are studied to determine the location, extent, and type of special drainage structures.

As to the advantages of this system of soil classification they appear to be many.

In many States, and in some of the Canadian Provinces, this classification has already been made for agricultural purposes. A vast amount of information therefore, merely awaits the awakening of highway engineers to a realization of its utility.

Until quite recently it was commonly believed that the highway engineer was interested only in those soil properties which affected the serviceability of the pavement. The modern conception of highway engineering, however, embraces the subjects of landscaping, erosion control, shoulder stabilization, roadside development, land use, etc. If the design

division is to handle all these subjects intelligently and efficiently with the data which can be furnished by the usual staff of soils engineers, it is imperative that a system of soil classification and soil mapping be adopted which is simple, and which automatically records a large amount of detailed information about each soil type without the necessity of a large amount of time consuming laboratory work. The pedological system of soil classification supplied this need.

The mere recording of the name of the soil type on the soil map of a given area, automatically designates the soil properties and the characteristics not only of the surface layer of soil in the area, but of any portion of the soil profile to any depth of interest to the highway engineer.

In addition to serving as a basic unit for mapping, the soil type serves as a unit to which laboratory test results, road behavior observations, construction problems, and maintenance experience, are related. Consequently, the soil type furnishes a medium for classifying and coordinating the accumulated soil experience of the personnel of the entire highway department.

As an example of the utility of the pedological system of soil classification, consider the alternative value to the designing engineer of labelling a given soil area as A-3 according to the Bureau of Public Roads system, or "Bellefontaine" in that particular area, according to the pedological classification, when the simple recording of the name "Bellefontaine" on the soil map of that area automatically implies the following information essential to design:

(1) Well drained sand and gravel.

(2) The water table is at such depth as to be safely disregarded in design.

(3) The grade line may be established at any point in the soil profile.

(4) No sand sub-base required for pavements.

(5) No subgrade mat required for gravel.

(6) Two hundred lineal feet of material of frost heaving texture to be excavated per one thousand lineal feet of cut.

(7) Three hundred lineal feet of six inch tile edge drain required per one thousand lineal feet of cut.

(8) Not necessary to construct fill in one foot layers.

(9) Shoulder stabilization not required.

(10) An admixture of binder is required for controlling water erosion of slopes.

(11) Slopes are not eroded by wind.

(12) Is available as borrow material.

(13) Five to fifteen percent shrinkage wnen used as borrow material.

(14) Soil is satisfactory for fills in swamp construction.

(15) Proctor technique is not required for the construction of high fills with this soil.

(16) Soil is satisfactory for shallow fills.

(17) Not suitable for stabilization on sandy shoulders.

(18) Not suitable as top soil on subgrades.

(19) Possible source of gravel.

In concluding this section, it should be emphasized that it is not claimed for the pedological classification that it can be used to provide the answer to such special problems as for example the design of stabilized gravel bases. For this and other special problems the soil tests developed by the Bureau of Public Roads are required. Stockstad states that Hogentogler has suggested to him verbally that the individual State Highway Departments use the Bureau's soil grouping of A-1 to A-8 in a general way, but at the same time each State develops a more detailed system to fit its more extensive needs. It is for this latter purpose that Stokstad has endeavored in his paper to point out the excellent possibilities of the pedological system of soil classification.

(c) **Soil Stabilization.**

Two papers were contributed on progress in research on soil stabilization. One by Dr. Winterkorn of Missouri on "Progress in Research on the Improvement of Soil Properties by the Use of Chemical Admixtures," the other by Professor Crandell of the University of Illinois on "Progress in Research on the Improvement of Soil Properties by the Addition of Granular Materials, Tar, Asphalt and Portland Cement."

Progress in research on soil stabilization will be taken up under the subheadings:

(1) Chemical Admixtures.
(2) Granular to Cohesive soils and vice versa.
(3) Tar.
(4) Asphalt.
(5) Portland Cement.

(1) Chemical Admixtures

Soil materials contain many components. Minerals, organic matter in various stages of decay, salt solutions, and gases, are some of the constituents found in soil in all variations of quantity, quality and state of subdivision. The physical and chemical character of any given soil depends upon the properties of its constituents but is not a simple additive function of these.

The chemical character of the surface of soil particles particularly in clays, governs the soil-water relationship, and thereby governs the swelling, shrinkage, permeability, binding qualities, and other properties of the soil. Therefore, in order to appreciably change the engineering properties of a soil, it is essential to have some understanding of the chemical character particularly of the clay and colloid fractions of a soil.

The affinity of soils for water increases with increase in base exchange capacity and with increase in silica-sesquioxide ratio and vice versa. While the x-ray is capable of furnishing valuable information as to the chemical and physical structure of clay particles, progress in this direction is slow, and for some time to come, measurement of the silica-sesquioxide ratio and of base exchange capacity, and the determination of the kind of exchangable ions, will furnish the most reliable practical guide to the possible water affinity of a soil.

Winterkorn also believes, from electrostatic considerations, that since clay particles almost invariably carry an electric charge, the water affinity of a soil is a function of the geometric form of the surface of clay particles.

He believes the thinnest water films occur on clay particles having a convex, nonuniformly-charged, surface, and the thickest films on particles having a flat, uniformly-charged, surface.

Soils may be either cohesive or entirely lacking in cohesion. When water is introduced into a dry, cohesionless, sand, gravel or silt, it first spreads over the entire surface of the particles of these materials to a film thickness which varies with the affinity of each particle for water. Water in excess of this bridges at the points of contact between the particles. The particles are held together, and the granular mass given some cohesion, due to the surface tension of the water forming these bridges. When the pores of these silts, sands, and gravels, are completely filled with water, this surface tension effect is destroyed, and the result is an inundated, cohesionless mass of aggregate.

Winterkorn believes this surface tension kind of cohesion is the only source of cohesion in all soils ranging in size from gravel to clay, except in those cases where the particles are either non-uniformily charged or carry opposite charges, when an additional cohesive force is brought into play. This latter cohesive force is electrostatic in character, and is formed by an electric chain of oriented water dipoles and ions beginning at a charged point on one particle and ending on a point of opposite charge on another particle.

Winterkorn then proceeds to show how these factors of cohesion may be utilized in the stabilization of soils.

In combining soil binders with mineral aggregates, greater cohesion will be obtained by combining particles of uniform but opposite charges, or particles with non-uniform charges, since this encourages the electrostatic type of cohesion. Thus a podsolic or acidic clay adheres more tenaciously to limestone and other basic aggregates than to acidic aggregates such as quartz.

The question as to whether the use of a small amount of clay binder of high plasticity is preferable to a larger amount of clay having a small plasticity index is discussed.

Fine materials added to larger aggregates not only fill the pores, but are adsorbed on the surface of the larger particles forcing them apart. The thicker this adsorbed coating of fines, the more the behavior of the stabilized gravel when wet approaches that of the wet soil fines. Therefore,

the amount of fines added must be such as to not appreciably separate the coarser particles from each other.

Soil fines with a high plasticity index will improve the cohesion of a stabilized soil if a basic aggregate is used with an acidic soil binder and vice versa. Otherwise such soil fines may lead to decreased stability in wet weather.

For a base course, because of the presence of moisture in amounts which minimize the electrostatic type of cohesion and tend to cause swelling, it is desirable to use a clay binder of low plasticity index.

For surface courses, a clay binder of high plasticity index is desirable, if combined with an aggregate of opposite electrostatic properties, in order to supply greater cohesion in dry weather and to swell sufficiently to close the pores and prevent entrance of moisture in wet weather.

If clay binder and aggregate are of opposite electrostatic character, a higher plasticity index can be tolerated in the binder than under the reverse conditions.

As for the effects of the various salts now used in soil stabilization, while the effect on soils of strong solutions of them is not yet completely understood, Winterkorn summarizes them qualitatively as follows:

(1) Changing of the surface chemical character of the soil particles by the adsorption of ions from the salts.

(2) Greater surface tension cohesion in the stabilized soil because of the increase in the surface tension of water brought about by the presence of these salts.

(3) Equalization of moisture content in the soil due to the deliquescent or moisture retaining properties of the salts.

(4) A decrease in volume change upon drying due to crystallization of the salt.

(5) Decrease in swelling of the soil because of the presence of the salts in the soil moisture.

(6) Certain components of the stabilized soil may be more soluble in the salt solution than in water leading to the possibility of redeposition of this material as a cement.

Mention is also made of the addition in France of sodium silicate to calcareous soils for the purpose of forming a calcium silicate cement between the soil particles.

(2) Granular to Cohesive Soils and Vice Versa

Professor Crandell discusses briefly some of the research results reported in recent technical literature.

Reference is made to the studies of Burmeister at Columbia University in endeavoring to quantitatively determine grading density relations for aggregates.

He refers also to the twenty panels on soil stabilization prepared by the U. S. Bureau of Public Roads for exhibition at last year's meeting of

the American Road Builders Association. These panels, now available in book form, cover the field of soil stabilization very thoroughly.

At the June meeting of the A.S.T.M., Goldbeck gave his conception of the desirable characteristics of a stabilized screenings base as:

(1) It should transmit wheel loads to the sub-base without appreciable movement and with very little deformation.
(2) It should perform these functions under all conditions of weather and moisture.
(3) In case of partial failure due to unfavorable subgrade conditions, the stabilized screenings base should have properties which permit it to heal or reknit again.
(4) It should remain stable even after the evaporation of capillary moisture is prevented by a bituminous wearing course.

A further excerpt from Goldbeck's paper states that maximum stability under wet conditions would be expected.

(1) If the particles are so graded in size as to mechanically interlock and have only a small percentage of voids.
(2) If just sufficient fine cementing material is present to stick the coarser particles together.
(3) If excessively thick water films about the particles are prevented either by integrally waterproofing, or designing to exclude water.

Professor Crandell reports as follows, the results of some of Goldbeck's experiments on the grading of aggregates and the use of bituminous materials, when the stability of mixtures of these was determined on a circular track.

(1) Gradation was the most important single factor in bringing about stability under wet conditions.
(2) The use of waterproofing materials such as tar, cut-back asphalts, and emulsions is not effective unless the mixtures are dense and have some fine dust present.
(3) There is a certain optimum moisture content for these mixtures above which instability occurs.
(4) Best stability occurs when the aggregate is graded to give high density, and when sufficient bituminous material is present to make for low water absorption and furnish some resiliency to the mix.

(3) Tar

Authorities consider the following points important when stabilizing soil with tar:

(1) The necessity or desirability of modifying the grading of the existing soil.
(2) The grade of tar to be used in each mixture.

(3) The amount of tar to be used.

(4) The use of water for mixing.

(5) The moisture content for most efficient compaction.

Professor Crandell reports an opinion of Anderton, Research Chemist for the Barrett Company which seems quite pertinent, as follows: "Thus far there has been no consistent effort to standardize laboratory methods for evaluating stabilization, nor for determining the relative efficiency of various treatments, especially as applied to bituminous materials. There is no general agreement as yet on the relative importance of many factors in soil stabilization and on the interpretation of test data, with the result that various laboratories are using different methods for evaluating stabilization and determining the best treatment for a given soil. In the present stage of development all methods for studying soil stabilization with bituminous materials are open to criticism."

A reference is made to the use of the Brabander Plastograph by Rhodes of the Koppers Company for testing the plasticity of stabilized mixtures.

(4) Asphalt

Professor Crandell refers briefly to a number of the papers given at last December's National Asphalt Conference, on stabilization with asphaltic materials.

Weathers of Florida reports having used cutbacks, emulsions and powdered asphalt with flux oils, obtaining best success with cutbacks.

Tilley of Nebraska, as the result of both field and laboratory research, recommends the stabilization of five to six inches of sand when the subgrade is dune sand, and three to four inches when the subgrade is sandy loam. He has found it more economical to rectify soil gradation in most cases than to use special asphaltic materials, and that the amount and type of filler has the greatest effect in determining the quantity of oil required. Tilley prefers MC to SC oils for this work because their higher stability and cementitiousness permit the use of inferior gradings and high percentages of bitumen when necessary.

The soils treated by Reagel of Missouri frequently contain little coarse aggregate, and the bitumen added is kept low (7 to 8%) in order to avoid plasticity of the mix.

Professor Crandell reports some of his own experiments and observations concerning stabililization with bituminous materials. With tar, all specimens except those made with 7% light coal tar became brittle in a short time. He reports that emulsion stabilized specimens hardened on top and crumbled away while the lower layers were still soft and pasty. Similar observations in connection with emulsions were made on the road. This hardening of the surface while the interior is still soft has also been observed with cut-backs and road oils.

Winterkorn has added a few interesting remarks on bituminous soil stabilization in his paper. He states that bitumens must function as a cement for granular soils, but as waterproofing agents for cohesive soils.

He briefly discusses the influence of such factors as base exchange on the success of bituminous stabilization.

In a paper given at last December's meeting of the Association of Asphalt Paving Technologists, McLeod gave an account of several years research on soil stabilization through the waterproofing of soils by means of the use of a combination of activating and wetting agents with bituminous materials. The theory behind this work had its foundation in the field of ore flotation, the process by which most of the valuable minerals of the world are now recovered from their ores.

In connection with the use of emulsions for the stabilization of soils, from a perusal of the paper by Wace presented at the meeting of the Association of Asphalt Paving Technologists last December, and of the discussion following this paper, there would appear to be a considerable divergence of opinion as to the particular utility of bituminous emulsions for soil stabilization. Nevertheless the use of emulsions for this purpose is increasing in the United States.

(5) Portland Cement

Professor Crandell refers to some recent work on soil cement stabilization done by the Division of Highways of the State of Illinois. An experimental project was constructed in 1936. Samples containing 10% of cement by weight were superior to samples containing smaller amounts. Other observations from this experiment are:

(1) Care must be exercised to obtain representative soil samples for the purpose of supplying control data for a project.

(2) Equipment for preparing soil, mixing cement, incorporating water, and compacting the mix, should be such as to reduce time of processing to a minimum.

In a report to the Highway Research Board last December, of a laboratory investigation into the possibility of designing soil cement mixtures on the basis of the absolute volumes of soil and cement, Professor Housel concludes that the cement-voids ratio should be the controlling factor in the design of such mixtures. This conclusion is strongly supported by the results of wetting and drying, and freezing and thawing tests on cylindrical specimens of soil cement mixtures in which the ratio of cement to voids was varied.

Dr. Winterkorn states that an investigation has been started to discover any possible effect of the chemical composition of the surfaces of the particles of various soil constituents on the results of cement stabilization. Because of the chemical reactions involved in the setting and hardening of cement, he expects that the chemical character of the soil may have an influence on the effectiveness of the cement treatment.

From the brief survey just given, and also from the many papers appearing in the technical press, it is apparent that the stabilization of soils is the livest topic in highway engineering today.

The present state of flux of technical opinion concerning the various methods for the stabilization of soils just outlined, is well illustrated

by a paragraph from Professor Crandell's paper. In it, he states that—"If one talks to some of the representatives of the many producers of stabilization materials, he will find that there is a reluctance to make any definite statement concerning the positive stabilization of earth roads. There is considerable groping about in the dark as it were, and no one in a responsible position wishes to go on record as being able to produce certain and sure results."

It is obvious, therefore, that there is need for a great deal of further fundamental research in this field.

BITUMINOUS MIXTURES

This topic will be reviewed under the subheadings:

(1) Aggregates.
(2) Quantities and Types of Bituminous Materials.
(3) Adhesion Between Bitumen and Aggregate.

(a) Aggregates.

Mr. E. L. Wales, Materials and Testing Engineer for the Arkansas State Highway Commission has contributed a paper on "Progress in Research as Applied to Aggregates for Bituminous Surfaces." He states that the Arkansas Highway Department has not carried on a definite research program, and his paper therefore contains an account of the close observations they have made concerning aggregates on construction projects in the field.

Crushed stone and crushed gravel are used for open graded surfaces while bank and river gravels are used in dense graded construction.

For their open graded mixes, practically 100% of the aggregate is retained on a No. 8 sieve.

The gradation of dense graded aggregates is permitted to vary with the locality, within limits, in order to make use of local deposits with as little preparation of aggregates as possible. Since all surfaces are given a cover coat of sand, stone chips or pea gravel, there is little opportunity for observation as to which gradation functions most satisfactorily.

For open graded mixes, crushed stone aggregate has been limestone, sandstone, and syenite. No appreciable difference has been observed in the performance of surfaces laid with any of these aggregates.

The crushed gravels used for open graded mixes have not all proven satisfactory, gravels of apparently similar composition behaving quite differently. Wales believes the observed differences were not always due to the aggregate.

He reports one case of failure in an open graded aggregate as being due to the use of a chert gravel of smooth glassy surface texture. Laboratory tests showed this gravel to be hydophilic.

Wales states that there has been no evidence that any failure of a dense graded mixture could be attributed to the mineral composition of the aggregate.

The hardness of the aggregate is considered important in open graded mixes in order to withstand the roller, but is believed to be unimportant in dense graded mixes.

For open graded mixtures, at least 50% of the particles on each size of screen are required to be crushed because it is believed the angular particles are necessary for stability, and that bituminous materials adhere better to a fractured surface.

No crushing requirement is included in aggregate specifications for dense graded mixtures.

Uncrushed pea gravel has been found equal to, and in some cases superior to stone chips or crushed gravel for seal coating.

Little study has been given to the effect of absorption of the aggregate upon mixes. Most aggregates have shown an absorption range between 1½% and 2½%. Where absorption rose to 3½% however, a noticeable drying up of the mix was observed. The percentage of moisture allowable at the time of mixing, spreading and rolling, is based upon free moisture, that is total moisture minus absorption moisture.

The fillers used in dense graded mixes have been clay and silt from sources near gravel pits, and the fines present in the gravel. The fillers have ranged from friable silts to clays having a plasticity index as high as 25.

The fraction passing the 200 mesh ranges from 4% to 18%. These variations in filler content have given some trouble in determining the asphalt content, but no difference in surfaces laid with a wide range of quantities and types of fillers has been observed.

(b) **Quantities and Types of Bituminous Materials.**

Two papers were contributed on "Progress in Research as Related to the Quantities and Types of Bitumen to be Used in Bituminous Surfaces," one by Mr. L. C. Krchma of the Socony-Vacuum Oil Company, the other by Mr. Bailey Tremper of the Washington State Highway Department.

Krchma states that the choice of the type and quantity of bituminous material for any purpose is covered by numerous highway manuals and specifications, usually as the result of rationalization and deduction previous to research.

In discussing bituminous primes Krchma lists the functions of the prime as:

(1) To develop a bituminous film or layer with the proper physical and chemical properties to make it impervious to moisture and thereby protect the mat above it.

(2) To form a continuous bond between the subgrade and mat.

(3) To increase the effective thickness of the mat.

(4) To consolidate the loose subgrade when required.

The prime must be below a certain viscosity depending on the subgrade to give proper penetration, but if the viscosity is too low the prime will penetrate too far leaving a lean permeable primed surface.

In general R. C. materials do not penetrate well because of too rapid evaporation or absorption of the contained solvent. Consequently they tend to give a membrane type of prime which is easily injured. However, where the base is composed of very absorbent material such as scoria, an R. C. may be the only bituminous material drying fast enough to give a good prime.

MC and SC materials are the most suitable primes. Of these, MC is more generally used and more satisfactory because it sets up leaving the asphalt suitably distributed in the subgrade surface.

Krchma refers to the work of Stoddard in Iowa on measuring the evaporation of moisture through primes as evidence of the greater suitability of MC-1 for preventing the passage of water from the subgrade to the base course or mat.

It has been demonstrated that MC's have a greater adhesion for mineral aggregates than SC's, and are consequently less liable to displacement by water in the subgrade when used as primes.

Krchma further believes that MC's possessing high gravity, high asphaltene, positive spot characteristics, are superior to negative spot materials as primes because of their superior adhesion qualities.

In quantity, prime materials may be applied at a rate of from 0.1 to 0.5 gallon per square yard depending upon weather, subgrade conditions, etc.

For the mat or wearing course, MC and SC materials are used for dense graded road mixes, and MC or soft asphalt for dense graded plant mixes. MC's, RC's or asphalt cements of various consistencies are more generally used for the more open graded aggregates.

Both Tremper and Krchma believe that in general MC materials are preferred to SC's, although there are some notable exceptions.

Tremper refers to a recent paper by Buchanan in which a distinct trend toward heavier SC materials and light asphalt cements for plant mix is reported as opposed to the use of heavy MC materials. The principal reason advanced to explain this trend is that aeration to remove the kerosene type solvent is not necessary where heavy SC materials are used.

When MC materials are preferred to SC's the reason for the preference have been well covered at the previous Conferences and include better adhesion to the aggregate, greater stability of the mix, gradation of aggregates may be more tolerant, and the mixes are less critical with regard to oil content.

Because of the tendency of bitumens to harden in service, there is a tendency toward the use of higher penetration asphalts in high type pavements, and for the use of softer asphalt bases for cut-backs.

The use of cut-backs made from these soft asphalt bases is expected to reduce mat brittleness.

Because of the hardening of bituminous materials in service, Hubbard recommends:

(1) Use as soft an asphalt as practically possible.

(2) Use as high a percentage of asphalt as practically possible.

(3) Mix and lay mixes at as low a temperature and within as short a time as possible.

(4) Thoroughly compact mixes and use seal coats to prevent the access of weathering agents to the bituminous films.

As for the quantities of bitumen used in low cost mixtures, in spite of the various proposed formulae, it is generally admitted that the trend is toward abandoning formulae and relying on laboratory observations and the practical judgment of the engineer on the project.

Hot mixes are still proportioned on the basis of voids, and cold mixes by surface area. Hubbard has suggested the pore viscosity concept in an endeavor to develop a single method of proportioning the bitumen for both hot and cold mixes. Sufficient information is not yet available to put this concept into practice.

Tremper notes that in the construction of bituminous macadam the New England States use from ¾ to one gallon per square yard more bitumen than is used in the far West for the same thickness of surface. He believes that the simultaneous spraying of soap solution when the bituminous cement is applied accounts for the smaller quantity of asphalt required in Washington.

Tremper also believes that the quantity of oil used should possibly vary with the source of the crude and the refining process.

In spite of the recommendation from many authorities to have the asphalt content of the mat as high as practically possible, some localities prefer the lean mix for stability to be followed by a surface seal. It is rather likely that the thin bituminous films in lean mixes harden more rapidly than the bitumen in the thicker films in mixes with more normal bitumen contents.

Krchma gives a brief review of developments in materials for surface sealing.

The materials used for seals are RC's, MC's, SC's and soft asphalt cements, with the RC materials apparently predominating.

Reference is made to the study of seal coating materials by Professor Legault of Colorado State College. As a result of his investigations Professor Legault recommends a special seal of the RC-1 type, whose distillation residue is harder than 40 penetration. The desirable characteristics possessed by this special seal are rapid penetration, rapid drying, high strength and ductility, marked adhesion, and imperviousness to moisture. These characteristics are reported to have been verified by its successful use in Wyoming, Montana, and Iowa.

(c) **Adhesion Between Bitumen and Aggregate.**

Tremper and Erwin from Washington, and Winterkorn from Missouri, have contributed papers on this very interesting topic. They each present a study of the theoretical points involved, and critically discuss a number of tests which have been proposed to measure the relative affinity of aggregates for water and bituminous materials.

Tremper and Erwin briefly discuss a number of recent investigations on the subject of the adhesion of bituminous materials to aggregates, including some of their own work in this field.

They report a number of interesting experiments on stripping, and point out that in wash tests where so-called stripping does occur, and mixtures lose their coherence, only in some cases does the oil completely separate from the mix and float on the surface of the wash water. In most cases after stripping occurs, little if any oil appears on the surface of the wash water even though the recovered stripped fines contain only from .1 to .5 of the oil originally present. They believe the released oil exists in the wet mix in the form of minute globules, which are retained on the 100 mesh sieve used for separating the stripped fines from the rest of the mix in their wash test.

Winterkorn gave a very complete discussion of the adhesion problem in a paper presented at last year's Conference and has enlarged upon it in his present paper.

He lists the factors upon which the adhesion between a bitumen and aggregate depend as follows:

(1) The physical and chemical character of the mineral surface.

(2) The physical and chemical character of the bitumen.

(3) The time and condition of contact between the bitumen and mineral.

While the adhesion of bituminous materials depends upon their chemical composition, there is no method at present whereby the constituents of a bituminous material can be separated and identified.

In natural and carefully refined asphalts, the asphaltenes, carbenes, and acids, which seem to contribute most to adhesive qualities, are quite stable against the agencies of weathering. On the other hand, the constituents in some processed asphalts, and in some asphalts resulting from excessive refining, while exhibiting very desirable and even enhanced adhesive qualities, are at the same time highly susceptible to weathering.

In designing bituminous mixtures, it is most desirable to combine hydrophilic aggregates with a bituminous material showing good weathering characteristics. If the aggregate is hydrophilic, a more active bitumen may be considered, at the cost perhaps of decreased weathering ability, or admixtures of materials such as fillers or chemical reagents may be considered to permit the use of a more stable bitumen.

If the use of an asphalt with poor weathering characteristics is contemplated, good results may be obtained by designing for high density so as to prevent the access of destructive agencies to the bituminous material.

The tests for determining relative adhesion discussed, are the water asphalt preferential test, the swell test, and wash tests.

In testing a bituminous mixture for resistance to stripping of the bitumen from the aggregate, the authors of both papers on adhesion emphasize the necessity for sufficient time of contact between bitumen and aggregate for full adhesion to develop before any stripping test is made. This curing time appears to be necessary to permit the migration of the surface active molecules in the bitumen to the mineral surface and for their orientation in the mineral-bitumen interface.

In connection with the water asphalt preferential test, these writers point out that this curing time is not provided for, and that the results may therefore be expensively misleading. A filler which goes into the oil phase in this test is satisfactory, but one which does not do so is not necessarily a poor filler.

Tremper and Erwin observe that this test has the further drawback of being made on the fines only, whose properties may be decidedly different from the coarse aggregate.

The swell test, as improved by Stanton and Hveem, is a widely used criterion of the adhesion between bitumen and aggregate, and is made on the complete bituminous mixture.

Since a curing time of 24 hours is provided for in this test, the full adhesion between aggregate and bitumen is given an opportunity to develop before immersion in water occurs.

Tremper and Erwin believe that swelling is brought about by water replacing the oil film coating of the aggregate particles. They believe swelling is also influenced by the chemical nature of the mineral dust used as filler, since some minerals swell in water.

Disadvantages of the swell test given by Tremper and Erwin are:

(1) Stripping of oil from the coarse aggregate has been observed when the numerical value of swell was low.

(2) Successful bituminous surfaces have been constructed in Washington with aggregates which would not pass the swell test.

Winterkorn believes that the measured swell depends on the water permeability and water affinity of the mix.

He believes however, that a very hydrophilic filler in a mix subjected to the swell test, might swell in such a manner as to close the pores to the entrance of further water, and a misleadingly low value of swell be observed. Such a mixture would show the expected abnormal swell if exposed to water for a longer period of time. Nebraska has recognized this possibility and has extended the period of immersion in this test from 24 to 140 hours.

Winterkorn also believes that since equal amounts of swelling do not necessarily result in equal decreases of stability, and stability tests should be made on pavement samples both with and without prolonged exposure to water.

Tremper and Erwin appear to prefer the wash test to the others for measuring adhesion because of the closer similarity of this test to road conditions. They measure the percentage of the minus 100 mesh fraction of an aggregate of standard grading which as a result of stripping in the wash test can be washed through a 100 mesh sieve. Their technique is suitable for measuring the adhesion characteristics of either aggregate or bitumen. They also use the wash test for measuring the time rate of improvement of adhesion between aggregates and bitumens by making tests immediately after mixing and after curing at 140° F. for four hours.

Because of the better adhesion of MC-2 over SC-2 in these wash tests Washington prefers the cut-back to the slow curing bitumen for light surface treatments and road mixes.

Saville and Axon have found that previous soaking in water is necessary in some cases in order that the results of the wash test may be made to duplicate field observations.

Hubbard refers to the investigation of the resistance to stripping conducted by the Committee on Characteristics of Asphalts of the Highway Research Board as reported in the 1937 Proceedings of that organization. A modification of the Nicholson wash test was used and found to be far too severe for mixtures made with liquid bituminous materials. Hubbard states that further modifications of this test appear necessary to make it generally applicable, and that there is much room for improvement in quantitatively estimating the degree of film stripping that may occur in the test.

Winterkorn within the past year has concluded that the Riedel and Weber test, in which adhesion is measured by the use of alkali solutions of graded concentrations, is not satsifactory because the effect of the alkali solutions is different from water, and their effect on bituminous materials is inconsistent.

In order to improve the affinity between aggregates and bitumens, Winterkorn has been studying the effect of the synthesis of very thin films of artificial resins on the mineral surfaces.

McLeod has been able to increase the adhesion of aggregates for bitumens to such a degree through the use of activating and wetting agents, that wet aggregates can be readily coated with bituminous materials, the water being displaced.

From this brief review of the adhesion problem in bituminous mixtures, it should be apparent that while much investigation has and is being done, there does not exist a universally acceptable test method for evaluating the relative adhesion of mineral aggregates for bituminous materials and water.

BITUMINOUS MATERIALS

In this division, three papers will be reviewed. Two of these papers are on "Research Developments Related to the Service Behavior of Bituminous Materials," one being prepared by Mr. W. M. Carney, Chief Chemist, and Mr. F. R. Grant, Asphalt Research Engineer, both of the Lion Oil Refining Company, the other being prepared by Mr. Leroy M.

Law. The third paper gives an account of Progress in Research on the Testing of Bituminous Materials," and was prepared by Mr. Prevost Hubbard.

In the first part of his paper, Law gives a brief account of the growth of conditions which have led to the present increasing interest in the service behavior of bituminous materials.

Originally, the producers of asphalt constructed pavements under performance quarantees to municipalities. However, with the development of highway construction into a major industry this has changed. The producer has become largely interested only in production, specifications are drafted by the consumer, and the contractor obtains and incorporates into the road surface the cheapest asphaltic material which will pass inspection.

Whatever the advantages of this development of divided responsibility may be, they have been accompanied by a number of disadvantages.

In spite of the endeavors of the six committees of national technical organizations who are interested in the quality and service behavior of road and paving asphalts, there are no reliable criteria which may be specified to guarantee the serviceability of asphaltic road materials purchased, and coordinated research remains neglected because the division of interest has made it difficult to obtain adequate financing for the fundamental research required.

Into the previously more open specifications, certain so-called identification tests have been recently incorporated, based upon the satisfactory performance of some particular material. Such identification tests are at best but a thin disguise for the closed specification actually intended, and should be as much against public policy as actually specifying the preferred material by name.

These identification tests have led asphalt manufactures to resort to blends and various subterfuges in order that their materials will pass diverse inspection required in various localities.

Engineers and chemists long ago found the standard tests inadequate to insure quality and serviceability. In their endeavor to obtain satisfactory asphaltic materials, they modified old tests and created new tests on such a scale that when the Bureau of Public Roads and The Asphalt Institute in 1930 undertook a program of simplification of liquid asphaltic materials, they were confronted by over seven hundred current products required to meeet about two hundred specifications incorporating a hundred and nineteen tests.

An immediate simplification of products and test requirements resulted with the adoption of the distillation test, whereby liquid asphaltic materials were divided into the three classes of SC, MC and RC.

However, the distillation test is unable to tell anything concerning the serviceability of the distillation residues.

Hubbard states that while the six tests of flash, viscosity and distillation, on the original material, and penetration, ductility, and solubility in carbon disulphide, on the distillation resdues, have been generally

satisfactory for cut-backs, this is not true in the case of the SC's for which further tests to indicate probable service behaviors are required.

As the next important test in the transition from purely identification to quality tests, Law lists the Oliensis spot test. Together with Carney and Grant, he believes that the Oliensis test affords a direct insight into the heat treatment an asphalt has undergone during manufacture. Hveem states that some crudes which have not been heated will show the same characteristics in this test as overheated asphalts. However, the modified Oliensis test may eliminate border line cases, and in determining the degree of heterogeneity may explain the varying success of heterogeneous materials in highway construction.

The authors of all three papers refer to the possibility that Benson's coagulation test may be developed into an index of service behavior.

From the work of Traxler and Combs on the etching of bituminous surfaces to indicate the amount of time hardening which may occur due to a rearrangement of the colloidal structure of an asphalt, and from the reaction of the coagulated portion of the films to solvents as reported by Benson, Carney and Grant suggest that Traxler and Benson may be attacking the same problem from a different angle.

In connection with the breakdown of bituminous materials, Thurston and Knowles have shown that the asphaltenes are the most easily oxidized fraction of asphalts, and that oxidation occurs through the absorption of oxygen. The oily constituents are the least easily oxidized constituent. It is possible that the asphaltenes in asphalts are protected from oxidation by the resins.

Strieter reported the formation of water soluble products in asphalts exposed to light as being responsible at least in part for the failure of asphalt coatings.

A number of tests which have been proposed to indicate service behavior are discussed by Carney and Grant.

They believe that the "Fluidity Factor" is not a satisfactory criterion of service behavior. They state that the assumption upon which this test is based, namely that certain viscosity characteristics can be obtained by one and only one combination of constituents in an asphalt, is not correct because a low rate of change of viscosity with temperature may be due either to highly saturated non-polar compounds, or to the presence of colloidal matter.

They consider the role of sulphur in asphalts to be too little understood to make a sulphur clause any guarantee of the quality of an asphalt.

They believe that the curve formed in the ductility tension test of Grant and Pullar is characteristic for each asphalt and indicates a certain definite composition. They believe this test might furnish a rapid method for indicating the percentage of asphaltenes, resins, and oily constituents in an asphalt.

The desirability for developing a test which will evaluate the properties of the various constituents of asphalts, particularly the resins, is

suggested. They believe that if the resins extraced by the Furfurol and Duosol processes are representative, that the binding qualities of an asphalt reside largely in the resins.

Law describes his own experiments with the weatherometer for determining the service behavior of bituminous materials as discouraging, no characteristic breakdown occurring even after several months exposure.

Hubbard believes the most important progress in the testing of bituminous materials has been in the field of bituminous mixtures. He states that engineers are beginning to understand that the characteristics of the mineral aggregate with which a bituminous material is combined, is as important to the serviceability of the finished mix as the characteristics of the bitumen itself. The interfacial energy relations between bitumen and aggregate, and the proportions in which they are combined, may result in success or failure in service.

Hubbard gives a brief review of research on the mechanical, physical, and chemical testing of bituminous mixtures.

Mechanical tests are made on compacted bituminous mixtures. They are designed to determine the resistance of such mixtures to applied loads, and are considered to determine the ability of the mixture to retain its original form in the pavement under the load and thrust of traffic. Stability, compression, and shear tests, have been used, and all give information of practical value which can be correlated to service behavior. Skidmore, Vokac, Hveem, Hubbard and Field, and other investigators, have published the results of researches on these tests. A standard test is urgently required in this field in order that stability may be expressed in standard units.

Resistance to the cracking of pavements in service has been studied from the standpoint of characteristics of compacted mixtures at low temperatures by Rader, and Raschig and Doyle. From an impact and cross bending test, they conclude that at low temperatures a pavement should develop a low modulus of elasticity accompanied by a high modulus of rupture.

Krchma, in his paper, gives a brief but quite complete summary of criticism of these tests by Vokac, who found that the compressive strength, elastic limit, and modulus of elasticity, were most satisfactorily correlated with the service behavior of the bituminous mixtures with which he worked.

Hubbard believes the determination of density per cent of voids to be of value when proportioning the components of a mix. He believes it is desirable to determine the average size of voids in order to select the proper bitumen for the aggregate, since the bitumen carrying capacity of the aggregate is largely governed by the percentage and size of voids in relation to the viscosity of the bitumen.

He states that water absorption does not depend on percentage of voids alone, since some mixes of high voidage become sealed against water absorption at exposed surfaces when compacted.

He suggests that changes in the physical properties of bituminous mixtures under an accelerated weathering test may be made to yield as valuable information as any test on the original bitumen or on the bitumen extracted from such mixtures.

In discussing chemical tests, Hubbard states that no satisfactory method for recovering bituminous materials from mixes particularly cut-backs has yet been developed. He believes the effect of centrifuging upon recovered bitumen requires investigation, since certain constituents may be removed, and the recovered portion thereby not representative of bitumen in the mix.

He refers to the excellent work being done toward the development of tests for soil bitumen mixtures in connection with soil stabilization programs.

In connection with the improvement of the service behavior of bituminous materials, Carney and Grant suggest that this might be effected at the refinery by blending to obtain certain definite proportions of asphaltic constituents. This presupposes of course, that the engineer knows what proportions to specify from a knowledge of the effect of each asphaltic constituent on the finished asphaltic product.

The hardening of bituminous materials in service has been reported by many investigators. Some reliable test to evaluate the relative resistance of bituminous materials to undue hardening under service conditions is needed. Progress along this line which may lead to the desired test has been made by the recent researches of Steinbaugh and Brown, Hubbard and Gollomb, Skidmor and Abson, Nicholson, and Raschig and Doyle.

Tremper believes that the time has not yet arrived when resistance to undesirable hardening may be included as a specification requirement.

However, the Michigan State Highway Department have recently specified that the bitumen extracted from a pavement immediately after construction shall not have decreased by more than 50% of the original penetration, and the ductility of the recovered bitumen shall be not less than 40 cm.

Contractors not complying with this specification have been severely penalized.

Law suggests that to encourage research with the idea of simply developing some miracle quality or service test will not only fail to solve the problems which have just been discussed, but will probably lead to disappointment. To date, research accomplishments have results from the uncoordinated work of interested individuals with the result that there has been only a confusion of individual experiences rather than the development of broad basic principles. He states that active co-ordinated research awaits only the provision of adequate financing, and that this should be the responsibility of all groups concerned with the manufacture and use of asphaltic road materials.

RESEARCH APPLIED TO MAINTENANCE

Mr. F. N. Hveem of the Division of Highways of California prepared a paper on this rather difficult topic.

Hveem states that for the most part, maintenance methods and operations for the purpose of correcting existing faults or damage to bituminous surfaces, are the result of experience gained through trial and error, and have not often been made the subject of research as this term is used in the laboratory.

Viewing the matter broadly however, the application of research to maintenance indicates that in the end almost the sole purpose of research is to reduce the cost of maintenance. Even where research results in cheaper methods for achieving satisfactory results, the term satisfactory always implies durability and minimum maintenance expense, so that regardless of the particular problem investigated, the maintenance department is the ultimate beneficiary.

Hveem believes that the burden of trying out the new ideas developed by research most frequently falls on the construction engineer, sometimes involving increased cost. However, the maintenance cost may be so reduced by the innovation that the overall cost is lowered.

He believes that a discussion of the effect of research on maintenance can rightfully include a discussion of almost all research related to the construction of highways, and he discusses this under several headings.

(1) **Subgrade.**

While the recognition of the necessity for adequate foundations by highway engineers is now universal, many roads are still built on inedaquate foundations.

Hveem briefly reviews the necessity for drainage, and the beneficial effect of the various soil stabilization processes.

He believes that the penetrating power of water is greater in the vapour than in the liquid state, and that attempts to interrupt capillarity by the use of a layer of open gravel with large pores, may not be successful due to the passage of water vapour through the layer, and its condensation against the layer above.

He suggests that many failures of dense graded oil mix surfaces are due to water vapour rather than capillary water.

He has found that the sealing of dense graded bituminous mixes placed over wet sand has decreased the stability of these as compared with unsealed specimens. Even though the sealed specimens lost stability the increase in moisture content was only slightly higher than for the unsealed specimens. Hveem believes this checks observations in the field where a small amount of moisture entering a bituminous surface in the vapour state may cause more damage than larger amounts of capillary moisture. He suggests therefore that in considering seals, some considerations should be given as to whether more water rises annually from below than a mixture would absorb from rain or snow.

He draws attention to the often observed profusion of plant growth on the sides of many oiled highways, which may be due to the presence of an excess of capillary water trapped by the impervious bituminous mat. He suggests a possible research problem in the development of an ideal plant with a well developed root system for gathering moisture, and with the type of foliage to rapidly dissipate it, as a measure for stabilizing the subgrades of existing roads where reconstruction is not possible.

He mentions another type of mat failure due to subgrade conditions where the cause is rather obscure. Certain subgrade soils appear to possess such a degree of resiliency, that the unusual bending of the pavement slab under passing loads induces abnormal stresses resulting in waves and ripples in the pavement. He believes that measurements of amplitude and frequency of slab movement would throw light on certain types of pavement failures.

(2) **Base Courses.**

Hveem believes the most frequent source of trouble traceable to base courses is inadequate thickness. Another is the use of base courses of clean rock without fines over subgrades which are plastic when wet. Mud works up into the voids of such a base course destroying its usefulness.

The continued use of the cementing value test in the construction of base courses is outmoded he believes, because of the tendency of the test to encourage the use of clay binders which may lubricate the stone particles when the moisture contents are high.

In placing graded crushed rock or gravel base course the tendency of the material in these to segregate during construction may be avoided by the use of a slurry base, in which the aggregate is premixed with a controlled amount of minus 200 mesh dust, and sufficient water to insure its thorough distribution. This permits ready compaction when the mixture is spread on the subgrade.

(3) **Aggregates.**

Hveem states that ideas concerning aggregates have changed, and that investigations of existing dense graded mixtures indicate that hardness of the rock is of minor importance for these.

Much costly maintenance has been caused in the past because of the failure to recognize that many mineral aggregates were not suited for bituminous construction due to the tendency of bitumen to strip from them in the presence of water. Research has recently pointed this out and protective measures are being proposed. In the meantime, the use of the various stripping tests for identifying such aggregates has brought about appreciable savings in the cost of maintenance.

In general, limestone, basalt, and trap rock, are satisfactory, while it is well, to view aggregates containing appreciable amounts of quartz, chert, and rhyolite, with suspicion.

(4) **Bituminous Materials.**

Most asphalt technologists agree that the asphalt content of a mix should be as high as practically possible. However, there is a wide difference of opinion as to how much is too high. Undue emphasis on pavement stability when proportioning the asphalt content leads to mixtures which are too dry and brittle, while the effect of too much bitumen is too well known to require further comment.

Hveem has observed that asphalt pavements constructed from twenty-five to thirty years ago are relatively free from cracking or failures, and that this cannot be said about many recent pavements. He suggests that modern refining processes in producing lubricating oil and gasoline may be removing certain essential protective colloids, and thereby resulting in less stable asphaltic materials.

Hveem believes that the best bituminous material to use in any situation is the cheapest one which will serve the purpose, and that when this criterion is applied, there are very few cases for which slow curing oils are not suited.

(5) **Special Problems.**

Under this heading Hveem discusses the design of patching material. Since this material was at times unsatisfactory, California made an investigation of both satisfactory and unsatisfactory patching materials. They concluded that the aggregate must be neither too coarse nor too fine, and the oil neither too heavy or too light.

Hveem gives the gradations of four patching mixtures together with the recommended bitumen contents, which have been standardized as patching materials in California. He lists satisfactory grades of bituminous materials for these patching mixtures as SC-4, ROMC-3, and MC-2, California preference being in the order indicated.

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Conference on
Progress in Research as Related to the Construction and Maintenance of Bituminous Surfaces

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Mr. McLeod: In the discussion this afternoon we will follow pretty much the outline that was covered during the summary that was given. We will take "Soils and Soil Stabilization" first, followed by "Bituminous Mixtures," "Bituminous Materials," and "Reserach as Applied to Maintenance."

We will begin the discussion on "Soils and Soil Stabilization." The first topic will be "Soil Mechanics." The meeting is now open for a discussion of that topic.

Dr. Hans F. Winterkorn: Missouri

I saw in Mr. Housel's paper that there were two concepts of stability in the case of cohesive soils. One concept to which Mr. Housel refers is that which emphasizes the importance of shearing stability, or shearing tests. The other point of view is that of the school of Terzaghi, which emphasizes compaction. Now, I think that both schools have good evidence for their point of view, and I wonder whether one could not bring those two points of view together in one way or other, perhaps on the basis of the physical-chemical properties of the soils in question.

It seems to me that these two points of view may both be right, but that they concern themselves with separate phenomena in a continuous succession of phenomena which should be logically connected. I wonder whether Mr. Housel would say something about this point.

Mr. McLeod: That sounds like a very good question. Have you any reply, Professor Housel?

Prof. W. S. Housel: Michigan

The question that Dr. Winterkorn has brought up is one that has frequently been the subject of discussion in meetings of this kind, and during the last year or so I have had to answer this question several times. I think it is an excellent one for the reason that it appears to practicing engineers that when the investigators have differences of opinion and are not able to agree among themselves, it seems a rather hopeless task for the practicing engineers to get some practical benefit from soil mechanics research.

As Dr. Winterkorn indicated, there probably isn't so much difference of opinion regarding the subject as there is a tendecny to emphasize different phenomena, and I think this is strictly true with this so-called division into two groups, one emphasizing consolidation of the soil and the other emphasizing plastic flow or shearing resistance. With the help of the blackboard I will try to do what Dr. Winterkorn has suggested and bring out wherein these two differences of opinion may really be brought into substantial agreement.

Incidentally, I rather got the impression from the review by Mr. McLeod that there were two diametrically opposed points of view. That is not strictly so. I hope that I did not create that impression in my paper, because it isn't a matter of absolute disagreement, but rather a difference in emphasis on two phenomena. I know of no better way to illustrate that to you than to take an illustration from some of the practical problems of foundation soils and then to apply it to a bituminous road surface.

If we consider the settlement of a structure on a soft foundation soil, we can represent what actually happens in the field by a settlement curve which indicates the settlement of the structure with the passing of time. When the load is placed on an indefinite mass of soil and the settlement measurements are taken, there is an initial period of consolidation, or comparatively rapid settlement, which may be a few hours in the case of a test size of area or it may be a few years in the case of larger foundations, and it is during that period that consolidation of the soil as measured by the confined consolidation test is of major importance.

Now, after that initial period of consolidation has elapsed, one of two things is liable to happen. If the load placed on the body of soil does not exceed what I would call the bearing value of the soil, the curve should flatten out with the consolidation complete for all practical purposes. I make that qualification because of the fact that there will probably be some slow adjustment taking place over periods of time that could be compared to geological periods of time. Considering the life of structures, we are not too much interested in that smaller increment of settlement that would extend over many years, perhaps even more than the life of the structure, so for practical purposes we can say that if the bearing value of the soil is not exceeded the settlement will stop.

If, however, the load placed on the soil is in excess of that of the ability of the soil to develop the concentration of pressure, which depends primarily on shearing resistance, we have what is commonly known as plastic flow. As a matter of fact when plastic solids were first defined by Maxwell, back in the last century, the definition of plastic flow was "the continued deformation without rupture ,which progressed at a uniform rate with time."

This (indicating on blackboard), straight line relation between deformation and time represents what happens when we have plastic flow. Five years ago, I would say, the viewpoint on this subject that emphasized consolidation, took the position that the stresses or the

load necessary to cause failure in shear corresponded to rupture stresses, something much higher than would produce the slow flow of plastic materials. This viewpoint didn't consider that the amount of plastic flow could be of more than negligible importance, so they emphasized consolidation quite to the exclusion of plastic flow.

I don't believe that is true today, because all investigators in soil mechanics recognize that plastic flow is of importance in cohesive soils. In fact, Dr. Terzaghi, at the last International Soil Conference, said it was his observation that many of these settlement curves indicated that the settlement approached a uniform rate greater than zero, which calls attention to the fact that plastic flow must be considered. Therefore, the two schools are in essential agreement with respect to the fact that you must consider both plastic flow and consolidation.

Such remaining differences of opinion have to do with whether or not soil has a definite shearing limit, a definite yield value. In other words, there is still a difference of opinion as to whether it is possible for the soil water system to come to a condition of static equilibrium at shearing stresses less than the yield value which would correspond to no further settlement.

Perhaps a diagram on the borad would best show the physical phenomenon involved. If the load is placed on a bearing area such as indicated on the board in generally accepted theory of materials, (cohesive materials that have shearing resistance), the load-carrying capacity of that mass is dependent upon the maximum pressure which can be developed on an element of the mass, which for the two-dimensional case may be selected as a cube immediately beneath the bearing area. The ability of that particular element to carry the load apart from the confining influence of the surrounding mass is a function of shearing resistance.

In other words, before failure of that particular element can take place the shearing resistance must be overcome, and the definite criterion of failure based on the maximum shear theory, is that the difference between the two principal stresses, the vertical stress and the horizontal, is equal to twice the shearing resistance of the material.

When failure does take place, the failure planes are 45° planes of maximum shear which are indicated as diagonal planes in the figure. The failure actually develops into a penetrating wedge, which is indicated by full lines and which is pressed down with the bearing area. Then lateral displacement of the material takes place when the horizontal pressure is exceeded, causing

upward displacement in the adjacent element, so that by considering the total developed pressure occasioned by shearing resistance on the supporting element as well as the one immediately under the bearing area we develop a relationship for the total load that the mass will carry the strength of both elements depending upon the cohesion or shearing resistance.

Finally, solution of the problem depends very properly on whether or not we can measure a definite static shearing resistance of these cohesive materials, and that is where the suggestion of Dr. Winterkorn comes in, that the solution may go back to the physical chemistry of soils and the characteristics of the soil-water system. The suggestion is particularly pertinent, because any continued difference of opinion rest on that particular point. I would say that those who emphasize consolidation consider the soil to be a system made up of two separate phases of water and solid particles, and they do not regard the combination as material in itself. Therefore, they talk about the equilibrium of the soil-water system as if the water in the pores was not capable of sustaining any continued pressure; that is, it would follow the laws of pure hydraulic flow whenever concentration of pressure was put on it, and as the water flowed out there would be continued consolidation of the solid portions of the mass.

The ultimate answer depends primarily upon whether cohesion is due to surface tension of the water in the capillaries, which is called by Terzaghi "apparent cohesion," or whether cohesion is due to the electrostatic forces acting at the solid surface of the soil particles through the water molecules in the adsorbed moisture films between the particles.

I would say that at the present time the status of this problem is not entirely settled; that is, there are those of us who believe cohesion in such a soil-water system is a function of molecular attraction between the water molecules and the solid particles, and surface tension as we know it, that is the skin tension at the opening of the capillary, is just an analogy and not a reality. That is the position I would take on that very basic question.

There are a great many others, on the other hand, who believe that surface tension furnishes a more accurate description of the equilibrium of this soil-water system. If surface tension is the source of cohesion, then when the soil is inundated and there are no surfaces at the openings of the capillaries to furnish that apparent cohesion, you must accept the soil-water system as a suspension of solid particles in water which could not have any positive static shearing resistance.

I have one suggestion that might be used as something of an argument and to me it provides a satisfactory answer. If you assume that the surface of a mass of saturated clay was subjected to rapid evaporation you would bring into play the full potential force of capillarity which may, in equivalent figures, amount to a good many atmospheres—a tremendous pressure, in other words. Dr. Terzaghi has said that the forces induced in that way are so great that the applied loads that are placed on the surface aren't even of the same order of magnitude.

Then consider that the evaporation is allowed to penetrate just a fraction of an inch below the surface. The theoretical surface tension forces should be entirely active, yet when the soil is loaded, if the surface evaporation is not allowed to penetrate more than a negligible amount below the surface, it has no measurable effect on the ability of that mass to carry the loads. It is possible to alternately wet the surface and dry it out, and so long as the change in moisture content is now allowed to penetrate it does not apparently have any measurable effect. This common experience is one reason that could be taken to indicate that these hypothetical surface tension forces are analogies rather than realities.

Mr. McLeod: Thank you, Professor Housel, that is very helpful discussion. Is there any other discussion?

A. E. Stoddard: Iowa
In connection with the design of a subgrade, I wonder if Professor Housel could explain at what moisture content the determination should be made to give a bearing power which could be used in the design.

Prof. W. S. Housel: Michigan
I'm sorry. I said that after talking about foundations I would say something about the design of a road surface. I neglected to do that.

Suppose that we were considering moving loads over a flexible surface with a clay subgrade beneath it. If consolidation is the major source of settlement, the loads traveling over the surface, spread out over a considerable period of time, would each consolidate the supporting soil a little, and it should come to definite equilibrium at some stage without having the surface fail as it does under rutting and displacement of the subgrade. If, however, exceeding the plastic limit, or yield value of the soil, as I have indicated, is the more accurate conception, then plastic flow and displacement might continue indefinitely without any tendency to achieve equilibrium after some period of time. Thus there is considerable similarity between the settlement of structures and the behavior of a flexible road surface, the only difference being that the bituminous mat is placed over the subgrade to distribute the applied load and re-

lieve the supporting soil of the high concentration of pressure at the contact surface.

Now, in answer to Mr. Stoddard's question, I would say that we could not prescribe, as a general rule, the definite moisture content for which the shearing resistance should be measured. Every soil has its own capacity to absorb water and change its state, and it is generally felt that the liquid limit represents the moisture content which produces negligible cohesion. It is also recognized that undistributed clays may have greater strength than the clays retain after manipulation in the liquid limit test. Thus in undisturbed clays the moisture content may be above the liquid limit and the soil still have some shearing resistance which can only be destroyed by working the clay.

It may be possible that the shearing resistance of every soil can be correlated with its capacity to absorb water and then if we could determine the annual cycle of moisture change for a given soil out in the field, we could estimate for example that in the spring of the year the moisture concentration was maximum, and shearing resistance would be decreased to a certain value. However, this would have to be done by a correlation of the Atterberg limit tests in the laboratory with the shear tests in the field.

Because of other variable factors and the present inadequacy of available data, the final answer to Mr. Stoddard's question is that the only positive answer is an actual measurement of the shearing resistance of the specific soil under each specific condition to which it is subjected.

Mr. McLeod: Is there any further discussion?

R. C. Schappler: Missouri

I have in mind certain uncompacted alluvail soils deposited in quiet backwater areas. The soil particles appear not to have assumed a close arrangement. In that condition the soils have a relatively large capacity for water and a corresponding low resistance to plastic flow. Is it not possible that the rate of loading could be adjusted to induce settlement but not plastic flow? In that way would not the gradually consolidated soil develop a greater resistance to plastic flow than shearing and bearing tests on the original material would indicate?

Prof. W. S. Housel: Michigan

If I got your question, Mr. Schappler, I think I could answer it in this way. When we run a confined consolidation test we can evaluate purely consolidation; that is, there is no opportunity for shearing failure to take place in such a confined consolidation test. When we go out in the field we must remember that when the load is placed on this material the only confining

influence that we have is that which is developed by the surrounding soil, which in turn depends upon the shearing resistance of the soil. We really have in the field unconfined tests and in the laboratory consolidation a confined test, and it is necessary, in my opinion, to evaluate that confining influence or maximum developed pressure which the soil can develop. It has been the experience in load tests and observations that I have been connected with, that that value is much lower than is ordinarily admitted, so that the opportunity for consolidation of the underlying strata to take place is definitely limited by that confining influence.

I didn't hear you well, Mr. Schappler. Does that answer you question?

R. C. Schappler: Missouri — Yes.

Mr. McLeod: Professor Housel mentioned some tests that were run by the Michigan State Highway Department to test his theory of design of flexible surfaces. You are probably familiar with his paper. I believe that the engineers present would like to hear from Professor Housel as to his ideas concerning the relative thickness of base course over clay subgrade and sand subgrade under say a three-inch bituminous mat. Would you care to discuss that, Professor Housel?

Prof. W. S. Housel: Michigan — I can give you my idea. I don't say that I will answer it. In the design of flexible surfaces where we have attempted to apply the conception of shearing resistance as the controlling element at least in cohesive materials, the problem is simply an extension of the one we have just been discussing, except that a slab, or structural element, is inserted between the applied load and the subgrade. It was our conception that the most direct method of evaluating the behavior of the mat was to measure the shearing resistance on the boundary planes of the bearing area, a punching shear, in other words, as the measure of how much load could be distributed laterally to relieve the element of soil under the central column, which we have always supposed is the element that fails first because it is the one that carries the concentration of pressure from the central column.

I won't discuss the formula at all; a number of you may be familiar with the previous article. The formula is set up on the basis that the thickness should be sufficient to supply shearing resistance on the boundary planes sufficient to reduce the applied pressure so that it doesn't exceed the allowable bearing value of the subgrade. In the load tests at the Michigan State Highway Department actual load tests were made on an oil aggregate surface, on a six-inch gravel base, over sand

subgrades and over plastic clay subgrades, and at a time of year when the moisture content was at its highest and the subgrade shear at its lowest. In the analysis of those tests it appears that the way in which the factors are separated out in the formula is a logical one. In other words, we are able to evaluate definitely the punching shear through the surface and by independent tests measure the bearing value of the subgrade. These two test values as combined in the formula check as closely as we would expect with the bearing test value taken from the load test.

In a general way, as an answer to Mr. McLeod's question about sand subgrades and clay subgrades, it is my impression that the standard practice of using, say, a six-inch flexible mat or a seven-inch flexible mat is perhaps a little more than is necessary on a well consolidated sand subgrade, and not enough for the clay subgrade which has a rather low shear value after the spring break-up. In other words, this road was actually failing over the clay subgrade due to traffic loads that were on it during that time of year, while over the sand subgrade it was standing up very well, and our tests indicated that the thickness could have been cut down over the sand subgrade and should have been increased over the clay subgrade.

Mr. McLeod: Thank you, Professor Housel.

I think we had better close the discussion on soil mechanics at this point and go on to a discussion of "Soil Classification," as presented in Mr. Stokstad's paper. Have we any discussion of this? It is a system of classification which is more or less opposed to that which is being developed by the United States Bureau of Public Roads, with which most of you are familiar.

E. L. Wales: Arkansas I have a question I would like to ask Mr. Stokstad. Does the pedological classification of soils indicate what pavement designs should be adopted, and to what extent does this classification serve as a basis for design in relation to the physical soil tests?

O. L. Stokstad: Michigan I think your question, Mr. Wales, involves the utility of the classification and its relationship to physical tests.

With respect to utility, it seemed to me as I read the papers prepared by the committee on Research, that the results of various investigators could have been more easily compared had their writings contained information relative to the zonal groups involved. For instance, it seems reasonable to expect that the characteristics of bituminous materials, (as indicated by test results) used on the Chernozems of Texas should differ from the characteristics of bituminous materials used on the Podzols of Michigan. In this connection I might suggest to

technicians in the bituminous industry that they develop different types of bituminous materials for use with the various zonal soil groups—a few products standard for the northern podzols and another group of products for the red and yellow soils of Mississippi.

Zonal soil groups are subdivided into soil series, the utility of which depends upon their intelligent use before any laboratory testing is done. For instance, last month the Michigan Highway Department was faced with the necessity of picking out a large number of projects for bituminous surfacing. Fortunately there were soil classification surveys available on practically all of these projects and it was possible from a study of the soil surveys to reject projects unsuitable for the type of surfacing planned. In other words, the projects were selected on the basis of the soils to be encountered.

To further illustrate utility, the Miami loam soil series, for instance, is an excellent source of clay material for stabilization purposes. The Plainfield series, on the other hand, is a source of sand for sub-base construction over Miami areas, and when occurring in the grade requires shoulder stabilization.

The pedological classification differs from some other classifications in relation to physical tests in that it is made before any testing is done. The classification being an indicator of physical and chemical properties, tells us where to test and what tests to run. The name Miami loam not only tells us something of texture, structure, drainage, etc., but also indicates a definite range in P.I. The Plainfield series on the other hand is a granular soil with a P. I. of 0 and would therefore not be sampled for tests designed to express the properties of cohesive soils.

The pedological classification supplies a great deal of general information for design purposes. It tells us what to expect from the various soils as foundation materials and how they should be treated. When specific information is needed about some special property, samples are collected and tests performed in the laboratory. The economy of classification, then, insofar as physical tests are concerned, lies in its ability to indicate beforehand where to test and what tests to make.

Does that answer your question, Mr. Wales?

E. L. Wales: Arkansas — Yes.

Mr. McLeod: Would anyone care to add anything further to this discussion of soil classification?

E. T. Lea: Mississippi — I would like to ask this question: From what Mr. Stokstad says, it seems that soils can be classified before testing. I don't understand that, exactly.

O. L. Stokstad: Michigan

By the term "testing" I mean laboratory testing. I suppose that the field work done in classifying and mapping soils could also be referred to as "testing." Testing in this sense consists of examining the soil by observing the environment and exposing the profile. The point is that routine laboratory testing is not necessary to classify soils according to the system used in the science of pedology.

R. C. Schappler: Missouri

I wish to endorse the idea of the pedological classification of soils as a means of helping the design department to prepare a reasonable plan for the construction of roads, taking into account differences in soil. The soil specialists with his group classifications and definite tests still is needed to take care of construction details and to correct features as they develop under construction.

The only criticism I can think of now of that classification is due to the soil scientists themselves having been concerned only with agricultural uses of soil. For instance, you refer to a silt loam soil, and you know that this silt loam is associated with maybe nothing but silt in the lower horizons. If you talk about, say a Wabash silt, that is all silt; whereas, the Putman silt has a very shallow top underlain by a very active clay. If you talk about the Wabash clay, the Wabash silt, and the Wabash sand, you have one series covering three radically different kinds of soil. That is rather confusing to an egineer.

However, I think the whole matter is a fine way of adapting what the soil scientists have done for our use, and thus broadening our knowledge of soils. For instance, any design man that has silt as part of his soil series will know that there are different names, but he will be trained to design for a particular subgrade. For instance, if you tell him he has gumbo he knows what he has to do, and that he has to take additional precautions on fill slopes.

The matter of pedological classification is mainly a question of the use of more standard names for various soils now known by local names, and familiarizing the engineers with the different series and types.

Mr. McLeod: Thank you, Mr. Schappler.

Prof. W. S. Housel: Michigan

Excuse me for intruding a moment, but the gentleman who asked about classifying the soil before you ran the tests on it, left an opening which I think is particularly good. The question was an excellent one.

I wanted to point out in that connection that it brings out the function of classification very well. After you have run the tests on a soil which have to do with design, there is no further need for classification. You

have the specific information to go ahead with your practical problem, so that classification becomes unnecessary when you have that information.

In other words, the function of classification is not to take the place of specific design tests; it is, as Mr. Stokstad says, primarily to give you a picture of your problem, and a general conception of soil properties but there is no substitute for measuring those actual physical properties which you must have when you get to the specific design problem.

Do you get the point there that I bring up? In other words, if we measure the shearing resistance of a soil we don't need to know the classification with respect to that property. We aren't interested in it any more, and at the same time, knowing its type, as in the pedological classification, there is no substitute for the shearing resistance test. We must make that anyway, because that could vary from very low shear to very high shear, depending on its moisture content.

Mr. McLeod: Is there any further discussion of soil classification? Is anybody else interested in asking a question, or making a further contribution?

J. W. Powers: Arizona

There is one question I would like to ask the gentleman discussing this particular problem and it is this: Are you speaking of soils which for all practical purposes have become static so far as placement is concerned, or are you talking about soils that are possibly drifting around? Take in the country from which I come, in prospecting the centerline with test pits we find there is such a conglomeration of soils that there is absolutely no way of taking any set of determinations you might have, or any set of soil classifications you might procure, and then expect to find them to be the same in the next cut from the one you have been prospecting in. In other words, I have seen many times where test pits have been dug in which you might run into comparatively heavy clay on the top layer; the next layer might be blow sand, the next might be sand and gravel. Then you go to the next cut and it might be absoluetly reversed. You might get your blow sand on top, your clay below, and you might not find gravel as far as you expect to go in that particular cut with the grade line.

Also, I have seen in sandy country around the Colorado River, where we have found a saturated clay intrusion below six or seven feet of sand. In that case it was the only one that we found on the project. If we had taken as a basis the material found in all the rest of the cuts we had been going through, we never would have found this particular clay intrusion sticking up into this one cut, so the problem we have is one in which the

soil at the present time probably doesn't remain static because of erosion conditions.

The question I am interested in is determining whether or not you have a static soil condition, and also whether the soil types you are working in are of an alluvial type or whether they are of gravel type.

Mr. McLeod: Do you mean static over a period of say two years or a hundred years?

J. W. Powers: Arizona — No, static over one hundred, two hundred, or a million years. In our case we get floods that may change the whole complexion of the country.

O. L. Stokstad: Michigan — It is of interest to hear, Mr. Powers, that Arizona also has widely varying soil conditions within a small area. The materials from which the soils of Michigan have developed are of glacial origin. Glacial drift is also usually characterized by wide textural variation within short distances. The geologist has found, however, in studying glacial formations that there is some order to the manner in which variations occur. This order is reflected in the soil classification with the result that a certain soil series indicates what may be encountered in the deep sub-stratum.

The Fox soils for instance, have developed on gravelly outwash plains which supply very uniform and excellent subgrade materials.

The Bellefontaine series, on the other hand, is similar to the Fox series except that it occurs on glacial moraines rather than on out-wash plains and for this reason, is characterized by wide textural variations in the sub-stratum.

Suppose that we have a proposed highway project on which there is a combination of Fox, Bellefontaine and Miami soils. The simple operation of making the classification, then, tells us where to look for trouble and what to expect. We know that the Fox areas will be trouble free. The Bellefontaine areas, occurring in rolling morainic sections, require careful checking for conditions causing frost heaving. The Miami series, being a plastic soil, presents problems entirely different. Design activities and construction procedures are regulated accordingly.

Michigan also has wind shifting soils in the form of sand dunes. When the Designer sees Bridgman sand on the soil map he knows that he must provide for the control of wind erosion.

J. W. Powers: Arizona — May I ask another question? In making this analysis, or soil survey, as you call it, how closely do you define the particular area in which you are working—50 feet, 100 feet, one mile, two miles or what?

O. L. Stokstad: Michigan

The soil survey consists of making a strip map for an area 200 feet or more in width. In most cases the soil boundaries are correct to within a few feet. Sometimes there are relatively wide transitional areas between one soil and another in which case two independent mappers might not agree as to the exact location of the boundary. Soil boundaries are, however, often controlled by topography or drainage and therefore easily located.

The proposed right-of-way is studied in detail in order that the survey may be complete. The distance covered in one day varies from 2 to 10 miles a day, depending on the complexity of soil conditions encountered.

Mr. McLeod:

I think we had better close the discussion on soil classification. I have had occasion myself to be very much impressed with the utility of the pedological system of soil classification as it has been so capably developed for, and applied to highway engineering by Mr. Stokstad and his staff of soils engineers in Michigan. Mr. Stokstad will be here during the rest of the Conference and I think it will be well worth while for any of those who are interested to talk to him individually.

The next section we will discuss is "Soil Stabilization." Are there any questions on this? It is rather a live topic and there should be a great deal of active discussion on it.

E. L. Wales: Arkansas

Mr. Chairman, under this topic I would like to ask about gravel base courses, and to what extent the physical soil tests may be used to select suitable gravel base course material? If such tests are used, what are the limits recommended by the members of the Conference? In these tests I refer primarily to liquid limit and plasticity index.

Mr. McLeod:

Does anyone care to discuss that? Maybe we could get Mr. Stoddard of Iowa to discuss it. I understand they have done considerable work on gravel bases in Iowa.

A. E. Stoddard: Iowa

I am afraid that we haven't very much information on this. I am not one of the members who prepared any of the papers on this subject. We have been using the Bureau's classification on our soils to a certain extent, and also making tests on the soils. I believe that so far as the binder is concerned the limitations on the liquid limit and the plasticity index are covered in the specifications which were recently promulgated by the American Association of State Highway Officials. I believe they were officially adopted this year. I don't remember the exact limits on those. I think in general the liquid limit of not to exceed 30 is the common value. There is a recommendation that the Bureau has been making that the plasticity index be from zero to 6, on

which I should like to hear some discussion as to what the plasticity index of zero means to any of the members of the organization. To me, when you get zero it doesn't mean anything. To me the range between zero plasticity and absolutely cohesionless material would seem to include a large range of material which would be usable but still would give exactly the same answer so far as the tests are concerned.

I am afraid I haven't any very definite answer to the question that the gentleman has asked, but I would like to ask this question on my own part.

J. W. Powers: Arizona

In connection with a plasticity index of zero, in our particular part of the country nearly all the base materials we use come from material that has been concentrated in washes. We usually find in that type of material that the P.I. will be zero as determined on the minus 40. That type of material is readily consolidated providing the grading is comparatively uniform from the maximum size used down to dust.

Ordinarily to set such material it is necessary to have somewhere in the neighborhood of between six and ten per cent minus 200, even though the plasticity index of the material may be zero. The setting effect of the silt combined with water will give you a substantial enough grade so that you can lay over the top of it and also mix on top of it, providing you keep it slightly damp during any mixing or laying operations.

In our country we get somewhat better results when the plasticity index is somewhere around 4. Ordinarily we pay no attention to the liquid limit, because in our case we run into materials that are very, very absorbent, particularly in what we call cinders, and that is used very successfully as a base, as is caliche. Ordinarily caliche will have a liquid limit somewhere in the neighborhood of 35, and cinders of about 50. The plasticity index of cinders ordinarily is zero, and of caliche anywhere from 6 to as high as 7.

A. E. Stoddard: Iowa

I believe Missouri has been building a good mileage of stabilized base. I wonder whether Mr. Schappler can give us any information in regard to this P.I. of zero, and what they think of it.

R. C. Schappler: Missouri

I think the discussion about the P.I. brings out one thing, that is to get the densest gradation you can manufacture is the main thing. That is the essential thing you are after. P.I. is supplemental to that; by introducing some cohesion and also having a little clay there that will better seal the very fine pores you do get the additional benefit from having the small P.I.

The main mistake we made in the early days was in

thinking that P. I. limits could be used up to as high as 18 or something like that, but as bases they are not very successful. We all subscribe to the lower limits on P.I., as I mentioned before.

A. E. Stoddard: Iowa — Have you had any sections that did not have a high P.I. that definitely failed?

R. C. Schappler: Missouri — Yes. They averaged around 12, or as high as 14 or 15.

W. H. Wood: Texas — Our specifications provide for a minimum of 50% retained on a ¼″ screen, the binder minus 40. Clay gravels have a P.I. not to exceed 12 and a liquid limit of 35. The caliches have a P.I. of 15 and a liquid limit of 45. We have gone back over practically all of our old roads and taken samples out of the bases and classified the jobs as to whether they were good, doubtful, or failures from the base standpoint, and upon testing the caliches we have plotted them showing per cent binder against P.I. and per cent binder against liquid limit, and we find those outside limits are satisfactory.

Dr. Hans F. Winterkorn: Missouri — It seems to me that since the soil binder materials differ from location to location, and since the aggregate differs from location to location, and then since the weather conditions (the amount of rainfall, the climatic conditions) change from location to location, that we cannot make a cut and dried limit for the plastic index which should be allowed. If limits are given, I think they only indicate that at different places between those limits good results have been obtained, but there still remains the problem for every state and for every kind of material, to find out which is the best. The limit doesn't say that with a P.I. smaller than 4 you always have to get good results. With certain materials a higher P.I. may be good; with other materials a lower P.I. may be good.

I think this base construction business is a matter of an art. It cannot be a science yet because we cannot measure scientifically all the properties of those materials. Therefore, we have to find what are the best mixes for our definite purposes on the locations where we have them.

R. D. Jordan: Alabama — I would like to ask if there should not be some relation been the liquid limit of the P.I. and the percentage of soil binder in the base mat. In other words, the gentleman from Texas, speaking of 50% above ¼″, probably has not more than 25% passing the No. 10. It seems to me (and it has been our experience in Alabama) that where the soil binder percentage is very low the L.L. and P.I. may go up, but where it is large it must be kept low.

W. H. Wood: Texas

We find that to be the case also. If the soil characteristics are what we term bad, we may still use that material as a base providing the percentage of soil is very low. If we have ten or fifteen per cent minus 40 material, we can go a little higher.

We also have used material, particularly caliche, which is entirely all that would go through the ¼". We realize that there are cases where that type of material has been used successfully. We find that if it has very little P.I. it has made a good road. We encounter that from time to time and use that under certain specifications. Standard specifications do not work, but when we find it, it is to our advantage to use it with a certain set of specifications to cover it.

In connection with the low P.I., I don't think a P.I. of 1 or 2 means much. We state in our specifications that if a material as produced within those maximum limits does not set, the contractor shall furnish sufficient approved binder to set the job well. I don't think you have any guarantee that any material will necessarily set within the lower limits of the P.I. We have had a good deal of discussion among ourselves with respect to that, to specify a minimum P.I. for material which is not to be topped. All of the roads I have spoken of are generally topped either with rock asphalt, which is highly impervious, or limestone rock asphalt, or a penetration of asphalt which in itself it very impervious.

Mr. McLeod: It is becoming late so we will bring the discussion on soils to a close and start on "Bituminous Mixtures." Is there any discussion on reserach as related to the aggregates used for bituminous mixtures?

J. W. Powers: Arizona

There is one question I would like to ask, and that is, what is the definition of stripping?

Mr. McLeod: Do you feel like answering that, Dr. Winterkorn?

Dr. Hans F. Winterkorn: Missouri

Well, I think you have me on the spot here to a certain extent. I think in certain locations we can say what is stripping. I remember some of the highway meetings. But, in general, I think that here is a need for a definition. We already differentiate between different degrees of stripping. In the wash test, according to Nicholson, as used in the Cooperative project of the Asphalt Institute and the Bureau of Public Roads, differentiation has been made between: No stripping, less than 25%, and more than 25% stripping, from visual inspection. We don't know what this "stripping" means, as long as we don't know all the physical and chemical factors involved.

Mr. Tremper, in his paper, says that very often when an aggregate was "stripped," it still had a very thin film of something which was still absorbed. The main mass

may have been stripped away, but there still was some constituent of the asphalt which was still adherent. Since this phenomenon is so variable, in a way, I think we need a definition of what is stripping, and I hope some committee will make it.

I have said something, although I did not answer your question. There is a definite need for doing it, but I don't know whether it would be worth while for me alone to make a definition without having it go further through the fire of discussion with some other people who have done important work in this field. Mine is only one man's opinion, but if certain representatives of this adhesiveness business get together and have a nice heated argument, maybe we will get the slags out of the ore and have the metal left.

J. W. Powers: Arizona

I had a very definite reason for asking that question, and I will explain now some of the reasons for asking the question. We have done some work on stripping and we are not satisfied that it means anything. We find in the materials that are found in our state that you do not get a definite stripping of the asphalt in that there is a true separation of the asphalt from the stone. What seems to happen is that you get bare places on the aggregate and an accumulation or thickening of the bituminous film on other areas, and it will not strictly strip from the aggregate as you would imply from the ordinary definition of such a word.

Dr. Hans F. Winterkorn:

We have observed the same thing, and I think Mr. Tremper has observed it also. What we have to do now is to study the effect which it has on the service behavior of a pavement. If some materials strip entirely clean, and some others exhibit contraction to droplets, and still others show bitumen concentration on corners and points of greater adsorption capacity. The question can only be answered by practical correlation. In order that we can us laboratory test results for specification purposes we do have to correlate them with actual field behavior of pavements constructed with the materials tested.

I am sure that for some locations and for some climates materials where the bitumen coagulates in drops but still sticks to parts of the aggregate will be perfectly all right. Even in severe climates, as in northern Europe, where they have a lot of rainfall and quite a range of temperature, if you make your system dense enough, put enough filler in it, your stripping may not be so bad or it may not have any practical effect.

This whole stripping business is a matter on one side of the chemistry of the interface between asphalt and minerals. On the other it is a matter of proper design.

Personally, I think you can use almost any bituminous material if you design the pavement properly for the aggregate and vice versa. Here again, a point arises in connection with Mr. Law's paper, where he said we should develop definite types of asphalt according to the raw materials available. That is, we should make "quality" asphaltic products according to the crudes from which we make them. Now, take our construction problems; while they are connected, in general, with a great number of phenomena, they usually present typical problems for a certain location or for a certain district or for a certain state. These we have to solve for themselves as specific cases; and that is really what an engineer is here for, because if we could make cut and dried methods anybody could use them. Engineering implies a lot of experience, in order that we can coordinate measurable things with the immeasurable experiences we have acquired in connection with the materials.

Mr. McLeod: Mr. Wales mentioned an absorption test and certain limits placed on it for controlling aggregates, or rather for controlling the amount of bitumen added to aggregates. I wonder if Mr. Wales will tell us just how they operate this test and what its utility has been.

E. L. Wales: Arkansas

The absorption test, as we make it in our laboratory, may be simple to many of you but it has given us a great deal of trouble. The coarser particles present no difficulty when we determine the absorption value, but when we go to the minus 4 particles we have difficulty.

We put the heat-dried fine material in a flask, fill the flask with water, agitate very rapidly, and then allow it to settle for 24 hours. We then determine the amount of water required to refill the flask to the original mark. We realize there is some error in this test, of course, as the period of manipulation causes an error that can hardly be measured.

We have not done a great deal of work on the absorption of aggregates and its effect on our finished jobs. Most of our materials have a total absorption, that is of the coarse and fine, of approximately 1.5 to 2.5 per cent. Most of the time these materials (I am referring now to road mixes with cut-backs) are shot when they contain at least the full absorbed moisture, and absorption doesn't seem to affect us a great deal, but during hot weather when we have used aggregates that have a total absorption of about 3.5 to 4.0 per cent there has been a noticeable drying up of our mixes under blading operations, and we have in some cases added a little additional asphaltic material so that the mix would look right. We did not try to determine this amount by any test or

formula. We merely added a sufficient amount of asphalt to make the mix workable. Such mixes on one or two small jobs have been very satisfactory—among our most satisfactory jobs.

H. G. Nevitt: Missouri

We have found that the water absorption is liable to be a false indicator of the tendency to absorb asphalt. I have one actual job in mind. The water absorption test reported to us (we didn't happen to run it) was 2%. The job failed, in fact, and had to be reworked, and we changed the test to the use of kerosene as a more likely indicator of what the asphalt absorption would be. We checked it with asphalt under heat and found it came out the same (I think the kerosene was 15 and the asphalt was 13) by prolonged heating of the asphalt, and we have since gone to the use of kerosene instead of water to make that test, as a safer measure of absorption in certain materials where the water does not seem to absorb due to the surface nature of the mineral, but the kerosene does indicate the asphalt absorption.

Mr. McLeod:

In connection with types and quantities of bituminous materials and bituminous mixtures, is there anything in connection with primes? I would like to ask Mr. Krčhma what he considers the advantage to be, if any, of positive spot materials over negative spot materials as primes.

L. C. Krchma: Missouri

Of course there are a variety of positive spot materials. Probably they vary a great deal more than negative spot materials, but there seems to be some rational reason for deciding that typical positive spot primes are more suitable and give you better results than negative spot primes. The primary reason is that positive spot materials exhibit better adhesion to soil particles, adhesion which is not lost in the presence of moisture.

The prime primarily is supposed to eliminate moisture from the mat, and consequently if you had a positive spot material which could not be displaced as readily as a negative spot material by water, as exhibited by work by Winterkorn and Kelly and Tremper, you could tell that the material was a better prime.

There are other reasons. You take a typical positive spot material; it is not as permeable to moisture. You take a film of positive spot asphalt (and there are some data to verify this, although not published) and it will be less permeable to moisture than a typical negative spot material. Of course, positive spot prime has the added property of penetrating better, giving you a better dispersion in the primed surface than you would have with a negative spot material, primarily because the positive spot material is more active. You get a more impermeable prime.

Mr. McLeod: Is there any further discussion on that point?

J. W. Powers: Arizona — In connection with the statement you made concerning positive spot prime, do you find any difference between those that have possibly a xylene equivalent of 30% and those that might have a xylene equivalent of 70% or even higher?

L. C. Krchma: Missouri — I am afraid that I can't answer that question. There is no information available, at least published, indicating that one is any better than the other. I don't know the relation between xylene equivalent and resistance to moisture. There is this to be said: I believe this was brought up by Mr. Grant, that you could get positive spot asphalts with a good deal of wax which did not exhibit any better adhesion and resistance to moisture than a typical negative spot material. That would be interesting research work to be carried out by interested parties.

Mr. McLeod: Is there anyone who prefers the negative spot material for primers for any particular reason?

H. G. Nevitt: Missouri — I would like to state that the negative and positive spot are merely classifications. A positive spot material is simply a different type, a different action material, and there may be a world of difference between the different spot materials. I think Mr. Krchma's remarks have always to be qualified with that thought, that negative spot materials are one type of asphalt. There are good and there the poor negative spot materials, and the same is true of positive spot, and we can't just say that all materials of either type are good or bad. It is merely a classification and not a quality test.

L. C. Krchma: Missouri — I would like to supplement Mr. Nevitt's remarks by saying that you can find materials (there is a recent paper being published by the Bureau of Mines) which are typical negative spot materials by the Oliensis test but have been cracked. Reducing the materials to asphalt, investigators are able to observe cracking which isn't noticed by the Oliensis test. It is difficult to classify materials, consequently, as cracked and uncracked materials on the basis of the Oliensis test. It would seem also, talking about positive and negative spot materials (and this is not an original thought on my part), that it is not rational to have the same specifications for the typical positive test material that you have for a typical negative spot, straight run material. The two asphalts are absolutely different, and it doesn't seem right that you should have the same specifications for both materials.

To get the best negative spot material you should have a typical straight run negative spot classification, possibly differing with the crude. For a good positive spot

material, although we can't describe the proper specification, there should be a specification which would give you a characteristic positive spot material, and consequently insure you a better material. Trying to supply both straight run and cracked material on the same specification doesn't seem quite right.

Mr. McLeod: Is there any discussion of the "Types and Quantities of Bituminous Materials for Bituminous Mats"?

F. F. Havey: Indiana

I would like to ask if there is any progress being shown on developing a formula for design of bituminous mats, with respect to how much liquid material to use. I would also like to ask another question, as to how different states handle the competition between different types of bituminous materials in liquid form, such as emulsions, cut-backs, and tars, if they have all three in competition.

L. C. Krchma: Missouri

Although this was briefly covered by my paper, I think this is a question for the floor to answer. I don't think we are in a position up here to decide. There is a general trend (and probably justified) toward the deciding of the proper proportion of bituminous materials in the laboratory. This may be going too far. I would think they could easily and profitably supplement their laboratory bitumen content with a formula which would take care of the variations they find in the field. This would be particularly valuable for road mix jobs. It is not safe to just determine the bitumen content on the basis of one laboratory experiment, and it costs too much to keep up with the slight variations. This might be possible.

The question still isn't answered. I think the members on the floor should be in a far better position to ask that question. The subject has been well summarized by Hubbard in the recent asphalt paving technologies, where he has found that proportioning of hot mix asphalt, cannot be done on the same basis as you proportion a low cost road liquid asphalt mixture. The low cost type bituminous mixes use formulas of which the New Mexico formula or the formula developed by Mr. Nevitt are quite typical; then, on the other hand, you have the hot mix, which depends on voidage. But finally, of course, comes the experience of the engineer. Hubbard has suggested the possibility of using the sizes of pores as a proper method of deciding proportion of bitumens, but he hasn't developed that phase as yet. He has just mentioned it as an excellent possibility of a method of attack.

Mr. McLeod: Does anyone car to discuss that further?

We will pass on, then to "Adhesion Between Bitumen and Aggregates."

A very good question there would be which one of the three tests, the water-asphalt preferential, the swell test, or the wash test is the best test of adhesiveness in the experience of the various states that are represented?

If there is no discussion, we will pass on to "Bituminous Materials and Their Service Behavior." Has anyone had any experience with the weatherometer—any successful experience? Mr. Law, in his paper, reported that he had tried this test and found he couldn't get any characteristic breakdown.

H. G. Nevitt: Missouri

I don't see how the weatherometer can be considered to indicate the life of roads. Just to go to some roofing experience which indicates that some of our oldest and best known asphalts for road construction the world over are relatively poor asphalts for certain roofing purposes compared to others, it would indicate to me that failure by light is not the failure that we have in the road, even though failure by light may be the failure that occurs in particular types of weathering on roofing. Therefore, I have felt since the time I have been familiar with that work, that the weatherometer, or any light failure test, will not be indicative of road failures.

I think the highway industry should either take the same position, or should correlate the weatherometer test with practice and come to some conclusions, because we often see the comparison made, but on the data that I have heard of and seen I just don't see how a weatherometer test would indicate the life of the material in road use, which is quite different from other uses.

L. C. Krchma: Missouri

The data published by Benson last year on the coagulation of films showed that he can get the coagulation on these films without the presence of light. It seems to indicate that that failure, if it is any indication, indicates that light is not necessary for failure of asphalt. The mechanism of failure of a highway is entirely different, I am sure, from the failure of a roofing. Probably Mr. Grant might have more to say about that.

F. R. Grant: Arkansas

I would like to say a little bit on that. I agree with Mr. Nevitt over here entirely, that it is probable that a road does not go to pieces through exposure to ultraviolet light. There is such a small portion of the asphalt exposed and such a small portion of the surface is asphalt that the effect of ultraviolet light appears to be insignificant. It is entirely different from what you have in roofing materials where almost the entire weathering surface is asphalt. Here the weatherometer is of great value in predicting the probable life of straight

asphalt. But even here I understand there is some question as to its value when fillers are considered.

In closing I would like to say that I endorse Mr. Nevitt's statement on highway work. There are other reactions such as oxidation, heat, etc., that play a much greater part in the destruction or deterioration of the bitumens in the road surface.

J. W. Powers: Arizona

In connection with the statement the gentleman just made a moment ago, that there is a comparatively small amount of the surface that is exposed, is it definitely known that there are no convection currents in an asphalt pavement?

Roland Vokac: New Jersey

In that respect it is believed by some at least that ultraviolet may affect if not actually penetrate the asphalt pavement. We know that ultraviolet does affect the character of the asphalt. The reason it is believed that ultraviolet can penetrate to such an extent that might be progressively damaging to a pavement is that ultraviolet can penetrate quartz. The greatest portion of an asphalt pavement is quartz, and penetration by ultraviolet is therefore possible and may cause loosening of surface particles due to the deterioration of the asphalt underneath the quartz particles.

In the asphalt pavement the surface is essentially a film of asphalt. But beneath that thin film which may become worn off by traffic is a grain of quartz, and beneath that grain of quartz is another layer of asphalt, and so on through the whole structure. By that I do not mean that ultraviolet alone could destroy the pavement, or that it does destroy the pavement; nevertheless, if, through ultraviolet as an agency, the asphalt in the surface of a mix is hardened to any great extent the stresses set up under the load may tend to cause such surfaces to show progressive cracking, abrasion, or deterioration from the surface downward.

L. C. Krchma: Missouri

It is rather unfortunate that Mr. Zapata is not with us. If I remember, he exposed films of asphalt to ultraviolet and found he had to have his films unusually thin, otherwise he didn't get a uniform transition. It seems the asphalt films on the quartz particles would be sufficient to filter out any ultraviolet before it got down very far.

Of course, in the work Benson did on coagulated films it didn't seem that you had to have ultraviolet light to coagulate the films. I am not inferring that this coagulation of film is a true mechanism of the failure of the asphalt, but it does seem to indicate that ultraviolet is not necessary for that phenomenon. It is difficult for me to believe that ultraviolet and actinic light would play too large a part in the failure of a road.

The asphalt films are too thick to permit it to penetrate very far.

J. H. Swanberg: Minnesota

During the last two years we have been conducting an investigation with 26 asphalts of 85-100 penetration and approximately 20 asphalts of 50-60 penetration in which we mixed the asphalts with Ottawa sand in such proportions as to give us a film thickness comparable with that of a sheet asphalt mixture. We agitated the mixtures in a mixer at a temperature of approximately 180 degrees F. We weathered these mixtures under the ultraviolet light and also without the ultraviolet light. Following the weathering of these mixtures for varying lengths of exposure from 22 hours up to 166 hours, we made specimens and tested them for tension, elongation, impact and abrasion, and found that where we had used the ultraviolet light we obtained a greater change in the asphalt than where we had not used the ultraviolet light. We also found very drastic differences among the various asphalts. We found that the relative positions of these asphalts after test were substantially the same, whether we used ultraviolet light or whether we did not, but we did find more change in the asphalts where we used the ultraviolet light.

L. C. Krchma: Missouri

Nobody doubts the fact that ultraviolet contributes to the failure of the asphalt. The question is, is it important in the mechanism we are building for the failure of bitumens in the road? Does it contribute to the failure of the road? If the ultraviolet light could reach the whole road there is no doubt that it would rapidly increase the failure of the whole road, but we doubt if it can. It is well proven that ultraviolet light does cause asphalt to fail, but does that come in here in this particular instance?

H. G. Nevitt: Missouri

Practical experience ought to have something to do with this. The ultraviolet on the high mountain passes is far greater than the ultraviolet light on ordinary altitudes. If so, all our high mountain roads in Colorado, Wyoming and California should fail more rapidly than they do on the level. I have seen a lot of mountain roads and I has seen no signs of that. I would judge this greatly increased amount of ultraviolet action to be found in high altitudes is not a factor in road failures.

Mr. McLeod:

Would anyone care to add anything further to this interesting discussion?

Dr. Hans Winterkorn: Missouri

Since both the Oliensis test and the Benson test have been mentioned it might be good to say something about them. It seems logical that an asphalt which tends to change its colloidal system (that means, which coagulates or acquires a different structure while in service), might lead into trouble in practical construction; therefore, the tendency to heterogeneity should be important. It is

very important how this tendency is to be measured. If we use the Oliensis test, which is very valuable for its purpose, then we add some material to the asphalt. This material which we add may change the conditions in the bituminous materials. We may induce a tendency to heterogeneity in certain types of bitumen. Therefore, if we want to know more about the practical side, some other tests which do not involve the addition of a foreign substance should be better, and this seems to be the case.

I think the Benson test can be much better correlated with practice than the Oliensis test. We had some bituminous materials which we studied rather extensively because we wanted to use them for our bitumen soil stabilization investigation, and these bituminous materials contained a number which showed postitive spots. On the other hand, we submitted these materials to Mr. Benson, who was kind enough to test them, and he found that, maybe of 4 positive spot materials, one was definitely bad under his test, another was intermediate, and 2 positive materials did not show heretogeneity or did not develop heterogeneity under his test, so it seems to me that the Benson test might be a better indicator than the other test, or that the bituminous system of certain positive spot materials, according to the Oliensis test, might in usage be just as stable as the bituminous system of a negative spot material.

Of course there is as great a difference between different types of positive, as there is between different types of negative materials; therefore, it is necessary if when we call a material heterogeneous, or inclined to heterogeneity, to state by what test it is measured. Personally, I think that some light came from Kansas. I think Mr. Benson's test has great promise.

Mr. McLeod: Thank you, Dr. Winterkorn. Is there any further discussion on bituminous materials? Is there any discussion on "Research as Applied to Maintenance," which is a pretty tough topic?

F. F. Havey: Indiana

As I recall the discussion before the recess, one of the topics in the paper on research as applied to maintenance problems was a discussion of patching mixtures, and I think if anyone could add anything constructive to that topic it would be worth while. I don't have anything to offer, myself.

Mr. McLeod: We don't want to get too deeply into the subject matter of the Maintenance Committee, but someone may have some interesting investigation of patching mixtures to report.

If there is no further discussion, gentlemen, I would like to thank you for your kind cooperation this afternoon.

Papers Prepared

on

Progress in Research as Related to Construction and Maintenance of Bituminous Surfaces

for

The Montana National Bituminous Conference

Progress in Research as to Applied Aggregates for Bituminous Surfaces

By

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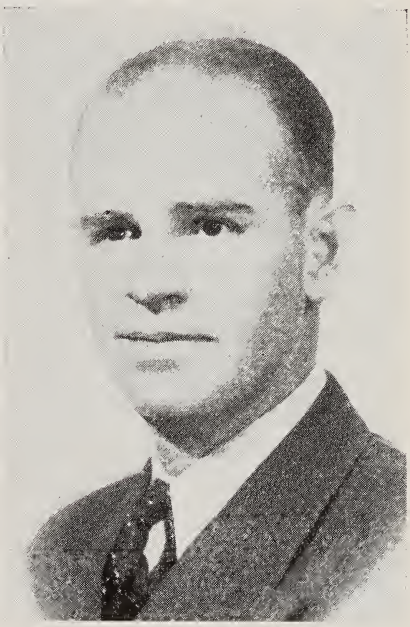

The Arkansas Highway Department has not carried on a definite research program, but we have given close attention to results obtained on construction projects, and the information given in this paper is from the observation and investigation of specific projects during the past eight years. This discussion of aggregates will be limited to those used in cold laid mixes.

Our construction has included both open-graded and dense-graded types, with the type specified for each project being governed by available materials. The bituminous materials used in the open-graded mixes have been rapid curing cutbacks and emulsions, while those used in the dense-graded mixes have been rapid curing and medium curing cutbacks, slow curing products, and emulsions.

The aggregates used in the open-graded surfaces have been crushed stone and crushed gravel with crushed stone predominating. Uncrushed gravel has been used in a few small projects for base courses only. The aggregates used in the dense-graded surfaces and bases have been bank gravel and river gravel with the bank gravel predominating. We have a job under construction at the present time in which crusher-run stone blended with fine sand and silt is being used, also a job in which sand of a gradation similar to concrete sand is being used.

As the aggregates used in the open-graded mixes generally have to be prepared, the gradations of these aggregates have been approximately the same. Below is shown the bracket within which the typical gradation occurs.

	Square Openings			
	1″	¾″	⅜″	No. 8
Total per cent retained	0	32-42	80-85	97-99

Results with this type of aggregate have been satisfactory, and there is not enough difference in gradation or results for comment.

Unlike the open-graded aggregates there has been a wide range in the gradation of aggregates used in dense-graded surfaces. The reason for

this is that local deposits have been utilized with as little plant manipulation as possible. Below are given the gradations of aggregates used on three jobs which represent the range of materials that have been used.

	Square Openings				
	1¼″	1″	No. 4	No. 10	No. 200
Total per cent retained (1)		0	31	42	86
Total per cent retained (2)	0		44	60	92
Total per cent retained (3)	0		57	72	94

Mixtures in which aggregates represented by gradations 1 and 2 were used give a very tight, dense surface, while mixtures in which aggregates represented by gradation 3 were used give a very open textured surface. There is a difference of opinion among engineers in this State as to which type is more satisfactory. Some prefer the coarser graded materials, while others prefer the finer graded materials. The writer prefers the intermediate type represented by gradation 2. There has been very little difference in the performance of surfaces built with aggregates of the various gradations. This is probably due to the fact that all surfaces are given a cover coat of sand, stone chips, or pea gravel. If the surfaces were left unsealed, there is little doubt that the finer graded aggregates, which give a tight surface, would prove more satisfactory.

The stone used in bituminous surfaces has been limestone, sandstone, and syenite. The gravel that has been used is composed of sandstone, quartz, flint and chert. As stated before, crushed stone and crushed gravel have been used in open-graded surfaces, and gravel only has been used in dense-graded surfaces, with the exception of the two jobs now under construction mentioned before. We have been unable to observe any appreciable difference in the performance of open-graded surfaces built with the various types of stone as all have been reasonably satisfactory. We have, however, observed some difference in results when gravel was used in both the open-graded and the dense-graded types. Some of our best open-graded surfaces have been those in which crushed gravel was used, while others built with gravel of apparently similar composition have developed failures within a short time, but these failures in all instances could not have been caused entirely by the aggregate used.

Our most unsatisfactory open-graded job, to which failure could be attributed directly to the surface course, was one built in the Northern part of the State last fall. The aggregate used was a chert gravel very angular in form with a very smooth surface texture almost glassy in appearance. In production approximately 50 per cent of the gravel was crushed. Failures developed within two weeks after the surface was opened to traffic. Most of the engineers connected with the construction of this job and others who have observed it are of the opinion that the failures were due to the mineral composition and the smooth surface of the gravel. Tests made in the laboratory indicate that this gravel possesses hydrophyllic qualities.

We have no evidence that failures in dense-graded surfaces may be attributed to the mineral composition of the gravel.

The hardness of aggregates used in open-graded surfaces is considered important because they must withstand the compression of the roller,

and proper keying action must be obtained, while the hardness of aggregates used in dense-graded surfaces is not considered important as stability is obtained by density, and a cover coat is used to withstand the abrasive action of traffic. All stone used in open-graded surfaces has shown a per cent of wear of less than 6, and all gravel used has shown a per cent of wear of less than 15. The per cent of wear of gravel used in dense-graded surfaces has ranged from 5 to 35. When soft gravel is used, a considerable portion of the particles on the surface break up under the roller, however, after the surfaces are sealed we have not observed any appreciable difference in the serviceability of surfaces constructed with hard or soft gravel.

As stated before, the aggregates used in open-graded surfaces have been limited to crushed stone and crushed gravel (a minimum of 50% of the particles on each size screen are required to be crushed.) Our reasons for this requirement are that the angular particles are necessary for proper stability, and we think that the bituminous material will adhere better to a fractured surface—the latter is questionable with some types of material. We have not included a crushing requirement in our specifications for dense-graded aggregates, and very little crushed material has been used in this type of work.

In the past we have required aggregates used in the cover material for seal coat to be stone chips or crushed gravel. During the past two years we have used uncrushed pea gravel for this type of work and have found it to be equal to and, in many cases, superior to crushed aggregates.

We built in 1935 a short job using an open-graded gravel base with on open-graded stone surface. The bituminous material used in both the base and surface was an RC-3 type cutback. The gravel used in the base was smooth, round river gravel containing no crushed particles and graded from a 1½" sieve to a No. 4 sieve. To date this job has required very little maintenance.

We have given very little study to the absorption of aggregates and their effect upon bituminous mixtures. Most of the aggregates used have an absorption range of from 1½ to 2½ per cent. In a few cases in which the absorption of the aggregate was rather high, approximately 3½ per cent, we observed a noticeable drying up of the mix. The percentage of moisture permitted by the specifications at the time of mixing and at the time of spreading and rolling is based upon free moisture. This permits mixing, and spreading and rolling when the total moisture content is less than the absorption of the aggregate plus the allowable free water.

The fillers used in our dense-graded surfaces have been clay and silt from sources near the gravel deposits and the fines present in the gravel in its natural state. These fillers have ranged from friable silt to plastic clays with a plasticity index as high as 25. The percentages of material passing the 200 mesh sieve have ranged from 4 to 18. These variations in the filler material have given trouble in the determination of our asphalt content, but we have not been able to detect any noticeable difference in surfaces laid with the wide range of quantities or types.

An interesting example of a dense-graded mix in which little or no filler was used is one section of the sand mix surface mentioned before

that is now under construction. This surface is being laid two inches thick on a six inch gravel base. This is a plant mix with an RC-4 type cutback. The sand is being run through a dryer and directly to a concrete mixer. The heat of the aggregate and the time of mixing is such that the final mixture contains little or no solvent. The mix is being delivered on the job at a temperature of approximately 225° F. The average gradation of the sand is as follows:

	No. 4	No. 10	No. 40	No. 80	No. 200
Total per cent retained	2	8	60	95	97

A part of this job has been under traffic for over two months, and there is no evidence of either shoving or rutting.

The percentage of bituminous material to use in dense-graded mixes has been a subject for debate in this State for some time. Numerous formulae from other states and agencies have been studied, but these give such a wide range in amount of bituminous material required that we have not deemed it advisable to base our mix design on any formula. The amount used on jobs to date has been determined by visual inspection of the mix. The amount of bituminous material used in RC and MC mixes is based upon the percentage of net residual asphalt in the total mixture. Those percentages on the various jobs have ranged from 3 to 5, and results with both the low and high percentages have been satisfactory.

Progress in Research as Related to Quantities and Types of Bitumen to be Used in Bituminous Surfaces

By

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Recent research on the quantities and types of bitumen to be used in bituminous surfaces appears to have been conducted by observation of results obtained in construction, rather than by laboratory studies to evaluate these variables. It is natural that this procedure should have been followed because of the uncertainty of interpretation of laboratory results. On the other hand, many other variables are, of necessity, introduced into construction work. As a result it is only through a preponderance of evidence as to results obtained that definite conclusions are warranted from construction data. It is not surprising, therefore, to find considerable differences of opinion between states as to which type of bituminous material is best suited for a given type of construction.

TYPES

Bernard E. Gray[1] has noted a rapidly increasing use of MC types of asphaltic road material. He attributes to kerosene cut-backs the ability to coat dusty or dense graded aggregates more readily and to displace moisture on aggregates to some extent.

J. W. Powers of Arizona[2] lists the following as advantages of MC products for plant and road-mix:

1. They develop a stronger binder than SC materials.
2. There is no appreciable loss of workability under favorable conditions during the mixing period.
3. They develop sufficient binding strength for low-filler-content aggregates.
4. The residual product is of rather soft consistency.
5. They are less susceptible than SC materials to displacement by water.

Mr. Powers notes the following disadvantages in MC materials, namely, that the mats are more difficult to rework after they have been down some time.

Montana[3] has found RC-3 in plant mix and MC-2 in road mix to be superior to SC oils.

Lawrence County, Illinois,[4] found good results in an MC-2 mat which was under traffic during unusually adverse weather conditions.

District 1 of the Bureau of Public Roads[5] selected MC-2 for road mix on the trans-mountain highway in Glacier National Park.

Washington uses both SC-2 and MC-2 in road mixes. MC-2 generally is more satisfactory. Its cost is greater than SC-2 and hence the latter is used to some extent on lightly travelled roads. Washington has constructed very little low cost plant-mix surfaces in recent years.

Opposed to the apparently growing sentiment towards MC materials in mixes is the State of Oregon. H. G. Smith, Construction Engineer, states[6]: "We cannot believe that MC-2 has any place in road construction as our laboratory tests clearly indicate that it is not possible for kerosene cut-back to be absorbed into the aggregate or to evaporate entirely from the mixture, and we are convinced that it affords a dangerous lubricating agent to eventually cause corrugations and traffic grooves." Oregon is now using RC-3 exclusively in road mixes and finds little difficulty in mixing during ordinary summer weather, particularly if the 200 mesh material is below 3 per cent.

Other states, notably Utah[7] and Colorado[8] report the use of SC-3 exclusively in road mix. Jackson County, Oregon,[9] used both SC-3 and RC-3. They report the RC-3 to be slightly more difficult to work. Livingston County, Michigan[10], uses both SC-3 and MC materials. They prefer the SC-3 because of easier reworking.

J. E. Buchanan[11] notes a trend toward heavier asphaltic materials in plant mix construction. Whereas SC-4, MC-4 and MC-5 have been used in the past, Nevada, Arizona and California are now using heavier viscosity materials such as SC-5, SC-6, and 151-200 penetration asphalt. Mr. Buchanan finds the chief advantage SC-6 plant mix over MC plant mix to be that aeration to remove kerosene solvent is unnecessary, hence it may be placed immediately by mechanical means without processing on the roadway. Mr. Buchanan notes that in road mixes, heavier viscosity products may be used if mixing is done with traveling mixing machines rather than by blade.

Missouri[12] has reported an experimental road in which powdered asphalt has been incorporated in mixes. They report that from 10 to 15 per cent of powdered asphalt was of distinct advantage in dense and medium graded mixes. They did not find, however, that powdered asphalt in any percentage up to 25 improved open-graded mixes.

For sealing, RC products are in greatest favor, RC-4 being used with cover stone and RC-1 and RC-2 when no cover material is added. Montana specifies a special RC-1 in which the base stock is of lower penetration and the solvent is a more volatile naphtha. Emulsions apparently are

used to some extent. Washington for several years used 151-200 penetration asphalt in two applications, each covered. Difficulties arising from adherence of the cover stone in cool weather have been responsible for a return to RC-4. Oregon used RC-3 with cover stone to seal road mix mats.

In the higher types of surfaces, bituminous macadam and asphaltic concrete, considerable experimentation has been done with asphalts of higher penetration including even the heavier SC products[11]. No formal reports of the behavior of these experimental roads have been noted. It is understood, however, that during the relatively short time they have been under observation, no lack of stability has become apparent. California[11] is using 101-120 penetration asphalt in asphaltic concrete on one project. Washington intends to use SC-6 (90 to 95% asphaltic residue) in place of 151-200 penetration asphalt, formerly used in bituminous macadam partly because of lower cost but also because the heavier asphalt has shown some tendency towards brittleness in extreme cold weather. On the other hand, Massachusetts uses 85-100 penetration asphalt for bituminous macadam.

The tendency to study the use of softer grades of asphalt apparently has been given an impetus from a recent realization of the extent to which asphalts harden during mixing and in service. It is important to note that there is little or no evidence to show that modern pavements are hardening to a degree any greater than those of older construction. The need for softer asphalts therefore will be proven not by the general knowledge that hardening takes place in service, but by a study of the conditions under which the harder grades have proven to be unsatisfactory.

It should be noted in this connection that the simple expedient of using a softer asphalt may not, of necessity, result in appreciably softer product in the eventual pavement. Lewis and Hillman[13] found that even some slow curing materials hardened to dangerously low penetrations under the accelerated exposure conditions of their tests. J. R. Benson[14] has found marked differences in thin films of asphaltic materials after exposure to natural and artificial weathering agencies. It appears that the characteristics of the asphaltic material may be fully as important as the penetration or viscosity at the time of use. While it is known that many users and producers are actively engaged in research on this problem it does not appear that we have yet arrived at a point where resistance to undesirable hardening may be included as a specification requirement.

QUANTITIES

The quantity of bituminous binder to be used in a given type of construction varies in accordance with the physical characteristics of the aggregate, atmospheric temperature at the time of laying and the type of construction equipment employed.

California in the construction of the East approach to the San Francisco Bay Bridge[15] increased the bitumen content of the asphaltic concrete pavement to provide greater flexibility on a subgrade in which further settlement was to be expected. The use of a richer mix was considered feasible because of the low seasonal fluctuation in temperature at that location.

Variation between states in the quantities of bituminous material used under similar circumstances is difficult to determine with accuracy, but in bituminous macadam there appears to be a wide difference in the practice of the New England states compared to that of the Western states. Specifications indicate that in the construction of a 2½ inch mat, application of asphalt to the base-rock amounts to from 1.75 to 2.0 gallons per square yard in New England against approximately 1.0 gallon in Washington and Oregon. Both areas use essentially equal amounts of asphalt in succeeding applications.

The New England States have a reputation for constructing excellent bituminous macadam and we in the West are proud of the results obtained here. It may be assumed that the quality of the surfaces is not radically different. The difference in the quantity of asphalt found necessary appears to result, not so much from differences in aggregate or asphalt. but in the method of application. Oregon developed and Washington uses a solution containing one per cent of soap applied through a spray bar mounted a few inches ahead of the asphalt spray. This solution is used at the rate of about 10 per cent of the volume of the asphalt. When the hot asphalt strikes the wet rock, vigorous foaming takes place, the foam sometimes rising to a height of a foot or more. After subsidence the under side of each aggregate particle is found to be coated with bitumen. It is thought that the use of the soap solution in this manner makes it possible to obtain thorough coating with from 0.5 to 1.0 gallon per square yard less of asphalt than would otherwise be required.

Little has been reported regarding the differences in percentage of bitumen made necessary by a change in the type of oil. Livingston County, Michigan,[10], however, reports the use of 0.52 gallon of SC-3 per square yard per inch of road mix mat against 0.60 gallon of MC material.

Apparently nothing has been reported as to the variation in the percentage oil required in identical mixes when oils of the same grade have been produced from different crudes or by different refining processes. There can be little doubt that a difference does exist especially as betweeen cracked and uncracked products. It would not be very difficult to evaluate oils on this basis in laboratory tests and the results might be of considerable economic importance.

Another factor influencing the quantity of oil to be used is the richness of the mixture desired on the road. Levi Muir of Utah[7] has stated the problem clearly as follows: "The optimum amount of bitumen is the largest percentage the material will carry and still retain sufficient stability." Mr. Muir follows with the statement that dense graded mixes, in particular, are sensitive to the amount of oil and lose stability seriously if over-oiled. He concludes that it is safer to work to the lean side and cover the mix with a seal. Similar conclusions apparently have been reached by the majority with respect to dense graded mixtures. In more open mixes the percentage of oil is not so critical with respect to stability. It is probable that practice leans toward a slightly richer mix with a view toward securing better resistance to water and abrasion.

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Progress in Research on the Adhesion Between Mineral Fractions and Bituminous Materials

By

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Definite advances in the understanding of the phenomenon of adhesion between aggregates and bituminous materials have been made the past year. Methods for converting hydophilic aggregates to those of a hydophobic character have been developed. The mechanism of adhesion has become clearer and methods of test have undergone further development.

Norman W. McLeod[1] has been able through the use of activating agents suited to the type of aggregate, and collecting agents, principally oleic acid, to coat damp No. 40 to No. 8 aggregates. He feels that dynamically stable mixtures which are not susceptible to stripping by water may be obtained. The practical application of this method would obviate the necessity of removing moisture from aggregates by drying. Costs of reagents range from 5 to 50 cents per ton of aggregate.

Winterkorn, Eckert and Shipley in their study of the Riedel and Weber test[2] found increases in adhesion of asphalt films when the aggregates were first coated with certain resins. A phenol-furfural resin gave a maximum adhesion for all the asphalts used.

In discussing Mr. McLeod's paper[1] Mr. Dow reported an improvement in adhesion through the treatment of aggregates with ferric chloride or with hydrochloric acid. Mr. Nicholson described effective treatments with iron and aluminum sulfate.

The activation of earths for bleaching purposes is carried out by acid treatment. According to Voigt,[13] this treatment leads to the increase of the activated surface by dissolving out oxides of iron and aluminum and replacing the surface calcium and magnesium with hydrogen or aluminum. Activated earths bleach mineral oils by adsorption.

Although his paper has been published for several years, the work of John M. Patek[3] is of such fundamental and pioneering nature that it is thought advisable to call it to attention here. Patek has investigated the "unwettability" (with water) of silicate mineral surfaces which were ground (but not polished) and then "cleaned of foreign material" by immersion for a few minutes in 6N hydrochloric acid followed by drying.

By measuring the contact angle made by a drop of distilled water upon this surface, the relative unwettability was evaluated. The angle formed by a drop of the floatation solution of water, sodium oleate and terpineol was also measured. High floatability seemed to be denoted either by a high contact angle with pure water, or by a large difference between an angle obtained with pure water and one obtained with soap solution. Floatability was found to be directly proportional to specific gravity and a function of the "valence ratio." By laboratory flotation tests 20 silicate minerals were rated with respect to floatability.

Saville and Axon[4] experimented with several variations of "wash" and "soaking" tests to determine the effect of pretreating a chert aggregate with proprietary oils and salts. Their results were not conclusive. It is noteworthy, however, that they did not find treatment with ferric chloride to be effective in promoting better adhesion with asphalt. Possible reasons for this will be discussed later.

The effect of ferric chloride in improving the adhesion of asphalt to aggregates has been investigated independently in the laboratory of the Washington Department of Highways. Improvement in adhesion by this treatment was noted by observation and measurement of the contact angle between asphalt and the treated aggregate surface in the presence of water. The results were checked and expressed quantitatively by the "wash test" as developed in our laboratory. Figure 1 shows the improvement in adhesion resulting from the treatment of a graded aggregate with various small amounts of ferric chloride before making a mixture with SC-2. This diagram shows a pronounced improvement in resistance to stripping of aggregate treated with as little as 0.03 of a per cent by weight (0.6 pound per ton) of ferric chloride ($FeCl_3.6H_20$). Satisfactory improvement of the aggregate represented by the top curve of Figure 2 was obtained by the use of ferric chloride in quantities considerably in excess of those required by the first aggregate.

We found this improvement, although Saville and Axon found no benefit from ferric chloride treatment. Assuming that the difference in character of the aggregate in the two cases was without influence, results appear to have been influenced by treating methods. We mixed oven dry (18 hours at 140° F.) treated aggregate, while Saville and Axon immersed the aggregate in a 10% solution for five minutes and air dried it for an indefinite period. We have found that when a drop of oil is applied to a surface which is coated with ferric chloride in excess of the amount required for activation and has been allowed to take up hygroscopic moisture, additional water easily undercuts the oil and removes it from the mineral surface to which it apparently never was attached.

We found by similar tests that the addition of benzoic acid or phthallic acid to SC-2 oil greatly improved its adhesion to silicate rock surfaces.

Thus it appears that adhesion may be enhanced through suitable treatment either of the aggregate or the asphaltic material.

Dr. Winterkorn stated in his paper at the 1937 Montana Conference that the first general attack on the adhesion problem appears to have

come to a successful ending, permitting at least the logical, although qualitative, exposition of the factors involved.

Adhesion has been defined as the resistance to separation of two adjoining materials. One of the fundamental requirements for the good adhesion of any adhesive cement is that the surface to be cemented is thoroughly wetted by the cementing medium. The film should be uniform and continuous. Fluid bituminous oil films, to be serviceable, must remain continuous under all natural conditions. These include the presence of water. With certain aggregates and some oils when first mixed, the continuity of the films under water is broken. After oiling, in the cases of some hydophilic aggregates, this condition exists a longer time than one might expect.

That adsorption is an essential part of our adhesion problem is now known. By adsorption the continuous flim eventually is held to the mineral surface by its attraction to its component molecules oriented at the oil-solid interface. This effect develops with time. Examples of the time required for orientation to take place are given in the research literature of lubrication and of flotation.[5]

An interesting discovery by Dr. I. Traube[6] is the adsorption of emulsified octyl alcohol and emulsified caprylic acid by surfaces which do not adsorb these molecules from solution to any extent. Similar work upon adsorption of oleic acid by minerals has been carried out by his colleague O. Bartsch,[7] who has shown that upon hydrophilic surfaces adsorption takes place in the same manner as upon more hydrophobic surfaces, but that the quantity held and the intensity of the force holding it are not so great as in the former case.* This immediately causes one to wonder if something similar takes place in the case of silicate minerals and emulsified SC-2.

One aggregate which appears from tests to be extremely hydrophilic in character has been investigated in considerable detail in our laboratory. This aggregate is represented by the top curve of Figure 2. The following results were noted in experiments on mixtures of this aggregate with SC-2 oil:

1. The fraction passing the No. 100 sieve and retained on the No. 200 sieve adsorbed 4 per cent of its weight of oil from an emulsion of SC-2 under conditions which must have been distinctly unfavorable to adhesion. The oil, though probably not in the form of a fluid film, was not easily removed by washing.

2. The 0 - No. 100 fraction when dry and mixed with 24 per cent of SC-2 resulted in a black, sticky mixture. When the mixture was shaken in water at room temperature for 15 minutes, no oil came to the surface. The oil, however, underwent a change as the mixture became yellow-brown in color, was non-coherent and could be washed through a No. 100 sieve. When water was added to the original mixture, and was shaken as before and then was heated to boiling, oil rose to the surface.

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*It is a very remarkable fact that a colloidal substance such as oleic acid is adsorbed by minerals, not in a single layer corresponding to Langmuir's theory, but in a great many layers. Bartsch showed microscopically that about 9,500 molecular layers of oleic acid were adsorbed on the surface of a mineral.

3. Dry aggregate, graded 0 to ¼", mixed with 6.4 per cent oil gave a normal mixture which was black, lustrous and coherent. This mixture, when soaked in water for 15 minutes, stirred, and then dried at a low temperature, became brown, "dead" and less coherent. A similar mixture made with wet aggregate, when dried at a low temperature became brown and "dead" and was deficient in coherence.

Differences in coherence of the mixtures were established by screening through a No. 40 sieve. Only a small amount of the "dry-treated" mixture passed this sieve in comparison to much larger amounts that were obtained from the "wet-treated" mixtures.

The following observations were made of the plane, rough-ground surface of this aggregate which was coated with SC-2 oil and then was immersed in water.

1. The oil contracted into small drops exposing the mineral surface.
2. Upon removal from the water, the oil spread out over the surface as it did before immersion.
3. These results occurred repeatedly upon re-immersion and removal from water. The drops rarely were detached from the surface.
4. A slight stain remained on the rough-ground surface after contraction of the film, indicating its former boundary. The stain appeared to consist of minute drops of oil around active points.

In this connection, the observation of the contraction of SC-2 oil drops previously spread on glass for 10 minutes, and immersed in immiscible liquids other than water is of interest. The contraction in liquids such as ethyl alcohol and glycerol is roughly the same as that in water. When immersed in aniline, which has virtually the same adhesion tension to glass as does water,[8] the oil drops spread out wider, rather than contract. The same occurs on a quartz surface. The interfacial tension of SC-2 oil against water is in the neighborhood of 15 dynes/cm., and probably much lower against aniline. The equation relating these tensions appears decidedly not to hold in this case.

The actual detachment of the contracting drop occurs in special instances. Alkaline water and probably boiling water will detach the drop.

It is not clear whether the stripping of oil from a mineral surface by water consists of few or of several manifestations. One may expect oil to come floating to the surface of the water after a stripping test has been performed. Some tests undoubtedly conclude in this manner. In those which include dust as part of the aggregate, and in which stripping takes place at room temperature, the water appears cloudy, and in extreme cases is very thick with suspended material, yet little, if any, oil appears at the surface of the water when the mechanical agitation is stopped.

Tests For Evaluating Adhesion

For purposes of research on the adhesion of oils to aggregates, or of hydrophilic or hydrophobic nature of aggregates toward oils, the recent literature has not indicated the use of the Water-Asphalt preferential test, to any great extent. Instead, other types of wash tests have been given

the preference. For research purposes, the water-asphalt preferential test limits the conditions which can be studied. It tests the 200-minus material only, which may have quite different properties than the coarser aggregate, especially as to the time required for curing. The oil used is limited to light grades, and the oil-filler mixture is not typical of practice. Due to the lack of curing data available, many believe the results of this test often to be misleading. As visual estimation is the usual method employed for evaluating the results, errors are easily made, for differences in settling volume or rate of settling may be very deceptive. Montana, however, has developed a method for recovering and weighing the separated dust.

The swell test, to which the above test was intended to be supplementary, is by far the most widely used for evaluating adhesional characteristics. Yet even here are limitations which need to be overcome. Differences in the penetration of water into the test specimen and the softening by water, are not disclosed by the numerical value of the swell. It is also difficult to study the effect of curing, particularly when cut back products are used.

Swelling appears to be brought about by the water replacement of the oil flim attached to the aggregate particle. It also may be influenced by characteristics of the mineral dust other than the adhesion tension for oil. Studies made on various clay minerals by X-ray methods[9] [10] [11] [12] indicate that certain crystals swell reversibly, in one dimension, with water. These minerals are also known to exhibit a large base exchange capacity. Prominent among such minerals is Montmorillonite, which is known to be a product of the decomposition of basalt in certain cases.[16] The structurally similar pyrophyllite does not swell with water and probably neither does kaolinite, and both exhibit but little base exchange capacity. Obviously, equal amounts of displacement of oil films by water may cause unequal swelling of the test specimens depending on the properties of the minerals present in the aggregate.

Stripping of oil from aggregate has been observed with coarse material taken from a swell test in which the numercial value of swell was low. The coarse material was taken from the cylinder, shaken a few revolutions with water in an Erlenmeyer flask and the mineral surface exposed.

A further objection to the swell test, in Washington, and one that probably has been observed elsewhere, is that successful bituminous surfaces have been constructed with aggregates that would be considered unsatisfactory from their behavior in this test.

Stripping tests which include the mechanical agitation or shaking of the oil mixture with water, appear to be in greatest favor because of their closer similarity to probable road conditions. The best known of these are the Nicholson and the Dow wash tests, which are quite similar. The Riedel and Weber test, which has been of interest in this country, has been investigated by Dr. Winterkorn and the results were published this last year.[2] Among Dr. Winterkorn's conclusions from this investigation was that the use of alkali solutions of graded concentrations as standards for the testing of the adhesiveness of bituminous films to

mineral surfaces is objectionable, because the effect of the alkali solution on the bituminous material is inconsistent and different from that of water.

The primary objection to the Nicholson and Dow tests is that the amount of uncoated material is estimated visually. The Nicholson test carries out the water stripping at 140° F. which is the extreme for natural conditions.

The wash test described briefly at the 1937 Montana National Bituminous Conference, and which has been in use in the Washington Highway laboratory for a year and a half is a modification of the Dow test.

This test involves the agitation in water of a mixture composed of bitumen and aggregate graded between 0 and ¼ inch. This is carried out at room temperature. After agitation a quantitative separation of "stripped" and "unstripped" particles finer than the No. 100 sieve is made by the simple method of washing the wet mixture through the sieve. "Stripped" particles pass the sieve readily while those on which the coating is unchanged are retained. The quantitative separation, then, is made on particles smaller than the No. 100 sieve only. The method assumes a similar effect on the coarser particles. The validity of this assumption may be questioned, but when the aggregate is prepared as is done in our laboratory, namely, by using crushed coarse aggregate only, it is felt that the action on the particles coarser than the No. 100 sieve is very much the same as on the fine particles. Visual examination frequently indicates that this is the case. If it is desired to test mixtures containing naturally occurring fine particles there may be a doubt as to whether "stripping" determined on the dust is representative of that of the entire aggregate. The test has the advantage over the water-asphalt preferential test, however, in that the action of the dust, not alone, but in combination with coarser aggregate, is observed. The test is considered superior to the swell test in that it involves agitation of the mixture in the presence of water. Other advantages are that a wide range of bituminous materials can be used, either the bitumen or the aggregate can be tested, and the preparation of the aggregate is standardized. An important feature is the ease with which the effect of age in curing the mixture may be studied. This is brought about through the use of small samples (50 grams) which can be brought rapidly to the temperature desired for curing. When the curing period has elapsed, agitation in water and washing are completed in a minimum of time.

The wash test has been in use in our laboratory for about 18 months during which time probably 300 tests have been made. We have found the results to be reproducible within reasonably close limits. We believe it to be sound in theory and that it measures the detrimental effects of water on bituminous mixtures.

In the above discussion of the test, we have used quotation marks in referring to "stripping." This has been done because the use of the term might be construed as meaning complete removal of the oil film. Although some light-colored hydrophilic aggregates appear visually to have lost their coating completely, we believe that actually a portion of the film is always retained. When the wash test fines have been dried first and then extracted with benzene, quantities of oil varying from pos-

sible one-tenth to one-half of the amount assumed to have been present originally are recovered. Not all of the retained oil is held with equal intensity because, after extraction with benzene, small but varying amounts can be extracted with an alcohol-benzene mixture, according to the method of Roehrick.[17]

Early in our studies, the question was raised as to what becomes of the "stripped" oil. We were unable to observe any sign of oil floating on the water despite the fact that the amount indicated as having been removed was sufficient to have been readily visible. It is now thought that the released oil exists in the wet mixture in the form of minute globules. These are retained on the No. 100 sieve in the wash test.

The separated oil presumably becomes attached to the aggregate surfaces again when the mixture is dried but the physical properties may have changed considerably. Earlier in this paper, experiments were described in which a mixture of SC-2 oil and a graded hydrophilic aggregate changed in color, luster and coherence when soaked in water and then dried at low temperature. The mixture undoubtedly became less serviceable for use as a road surface. Changes such as this are believed to be measured by the wash test.

Further evidence of the alteration of the surface characteristics of the fine material recovered from the wash test is that though alkaline solutions tend to strip oil films from aggregate surfaces, the adsorbed oil remains in place, and the fine particles are actually flocculated readily by sodium hydroxide solution. Hydroxyl ion is a dispersing agent for clays and silica[15] which therefore normally take a negative charge. Hydroxyl ion supposedly cannot ordinarily act as a bond between particles[14] but it appears that due to the adsorbed oil, the wash test fines may now be positively charged. Calcium chloride is a rather weak flocculating agent for these fines, and hydrogen ion acts as a dispersing agent. The flocculation by sodium hydroxide therefore would presume a very intense adsorption of hydroxyl ions by the positively charged wash test fines. The wash test, then, measures not only the removal of a portion of the coating from the surfaces of aggregates, but also the transformation of the coating to a state that reduces the coherence and serviceability of the mixture. Whether either or both phenomena take place with a given aggregate, the loss in the wash test is believed to indicate the extent of the impairment that may be expected when such a mixture is exposed as a roadway surface to the action of water.

Some tests show the percentage wash to be in excess of 100%. The recovery of fines in an amount greater than was present in the original aggregate is the result of grinding action of rather soft materials during the period of agitation in water. This is illustrated by the following data:

Sample No.	Treatment	Grams of graded ¼"-minus	Grams of 100-minus in agg.	Grams of 100-minus recovered	Per cent of 100-minus recovered
1	Unshaken	47.0	8.56	8.66	101.1
2	Shaken	47.0	8.56	10.22	119.5

Figure 2 shows the effect of curing mixes at 140° F. for various short periods of time. Our studies have indicated that curing for one

hour at 140° F. is approximately equal to curing for one day at 70° F. It will be observed that all the aggregates illustrated show an improvement against wash with length of curing time. This has been true for all aggregates tested. The rate of improvement, however, varies considerably among aggregates.

In our laboratory we evaluate aggregates on wash tests made immediately after mixing (0 hour curing) and after 4 hours curing at 140° F. If the 0 hour wash is below about 35 per cent, it is concluded that the material is distinctly hydrophobic and that it will give good results in bituminous work. When the 0 hour wash is higher, say 60 to 70 per cent, then the 4 hour result is considered. When the drop between 0 and 4 hours is considerable it is taken to mean that the bituminous treatment will improve relatively rapidly after construction. Of two aggregates showing the same 0 hour wash, the one showing the lower 4 hour wash would be considered the better. Of two aggregates showing the same 4 hour wash, the one showing the higher 0 hour wash would be considered the better because the rate of improvement after 4 hours would be expected to be more rapid.

We continue to make swell tests on all aggregates as additional evidence of their characteristics. Frequently the swell is in excess of the established limit, but the wash test indicates satisfactory performance. We then give greater weight to the wash test, a policy that is in line with our previous experience that the swell test rejects some usable aggregates. Whether or not this course will be entirely justified by experience remains to be seen.

The only distinctly bad aggregate, judged by the wash test, that we have tested is the one represented by the top curve in Figure 2. It shows a high swell also. It has not been used in bituminous construction. Those aggregates of doubtful quality, as judged by the wash test, have been rejected for use with SC-2 but, in some cases, have been used with MC-2.

The trend in Washington has been toward the use of MC-2 in place of SC-2 in light surface treatments and road mixes. It seems probable, more or less in retrospect, that among factors that have led to this change in type of oil, the ability of MC-2 to remain as a satisfactory coating on mediocre aggregates must have been of major influence.

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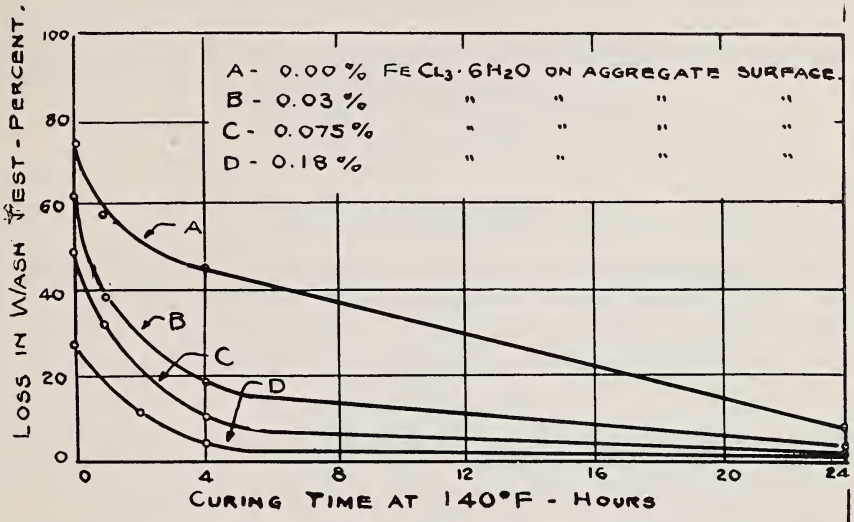

FIG. 1. TREATMENT OF AGGREGATE WITH FERRIC CHLORIDE.

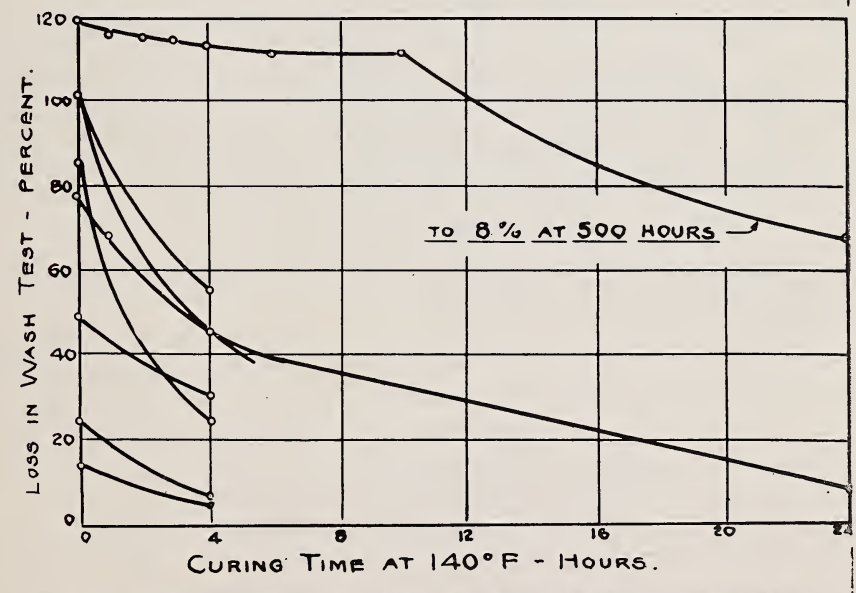

FIG. 2. REPRESENTATIVE WASH TESTS, VARIOUS AGGREGATES.

STATE OF WASHINGTON

DEPARTMENT OF HIGHWAYS

OLYMPIA, WASHINGTON

August, 1938

TENTATIVE METHOD OF CONDUCTING THE WASH TEST OF BITUMINOUS MIXTURES

(Revised July, 1938)

Description of Test

The wash test (Note 1) consists essentially in determining the proportion of aggregate in a bituminous mixture from which the coating is "stripped" during agitation with water under controlled conditions. Quantitative determination of the percentage of "stripped" particles is made on the portion of the aggregate finer than the No. 100 sieve. When the mixture is washed through a No. 100 sieve, fine particles from which the coating has been "stripped" pass through readily, while those on which the coating has been unchanged are retained. (Note 2)

Uses for the Test

The test may be applied either as a measure of the adhesive characteristics of the aggregate or of the bituminous material. When making tests of the aggregate, it is suggested that asphaltic road material, SC-2 be used and that a large sample be reserved as a standard for this purpose. When testing bituminous materials, a standard sample should be prepared from aggregates upon which coatings are rather readily "stripped."

Grading of Aggregate

The aggregate to be tested is screened and recombined to the following grading (Note 3):

		Per Cent	Cumulative Per Cent
	Retained on ¼" square	0	0
Passing ¼" square,	Retained on No. 4 sieve	10	10
Passing No. 4 sieve,	Retained on No. 10 sieve	25	35
Passing No. 10 sieve,	Retained on No. 40 sieve	35	70
Passing No. 40 sieve,	Retained on No. 80 sieve	12	82
Passing No. 80 sieve,	Retained on No. 100 sieve	0	82
Passing No. 100 sieve,	Retained on No. 200 sieve	6	88
Passing No. 200 sieve,		12	100

Coarse aggregate is crushed and pulverized to obtain fractions to give the above grading.

Mixing Process

A sample (Note 4) of the graded aggregate is warmed in an oven for six hours or more at a temperature of 130° to 140° F. (Note 5). Mixing

with the bituminous material is started immediately after removal from the oven. The bituminous material is also warmed to a temperature of approximately 140° F. previous to mixing. Materials that are too viscous for mixing with the aggregate at this temperature are cut back with naphtha or kerosene. (Note 6).

The warmed aggregate is placed in an earthenware bowl or porcelain mortar (Note 7) and the bituminous material is added in a quantity to result in a mixture of medium richness. (Note 8). The mixture is stirred and rubbed vigorously with a large spoon until all particles of the aggregate appear to be coated uniformly.

Preparation of Samples for Water "Stripping"

Two or more fifty gram samples are carefully selected from the mixture, (Note 9) according to the number of curing periods to be studied, and are placed in 6 ounce ointment cans (open). (Note 10). The sample for immediate testing (0 hour) in its container, is floated on the surface of a 77° F. water bath for about 5 minutes to insure cooling to that temperature. It is then transferred to a 250 ml. Erlenmeyer flask and 100 ml. of distilled water at 77° F. is added and the flask given a few brisk shakes.

This is to break up any large agglomerations of mixture. The flask is attached in a vertical position by means of a burette clamp, to the oscillating post of a Ro-Tap sieve shaker and it is agitated for a 10-minute period at 280-300 revolutions per minute.

Separation of Uncoated Fines

At the conclusion of the shaking period the contents are washed through clean No. 40 and No. 100 sieves (nested together loosely to allow air to escape as water flows through) by means of a fine spray of water, and the No. 100-minus solids are caught in the sieve pan. (Notes 11 and 12). The wash water and suspended matter are transferred to a 1-liter beaker. The heavier solids adhering to the pan are transferred with the aid of a jet of water from a wash bottle. A second washing is made, after which the water, this time containing no suspended matter, is decanted off, and the heavier sand particles transferred as before. A third washing is made of the material retained on the No. 100 sieve. The whole washing and transferring procedure will require approximately 7 to 10 minutes. (Note 13).

Flocculation of the Suspended Solids

From 15 to 30 ml. of a 10% sodium hydroxide solution is now added to the contents of the beaker in order to flocculate the suspended matter. It has been found that 15 ml. usually is sufficient, but occasionally as much as 30 ml. is necessary to obtain a complete settlement within 4 hours. In place of sodium hydroxide, a few ml. of a saturated calcium chloride solution is effective in causing complete settlement within 24 hours.

Filtration of Flocculated Solids

After settling, a large portion of the clear water is decanted, or, preferably, syphoned off. To the remaining 50-100 ml. containing the

flocculated solids, 2 or 3 grams (weighed to +0.005 g.) of filter aid is added. The mixture is filtered through a tared gooch crucible containing a thin asbestos mat. A small portion of the filter aid may be placed in the crucible and washed through to form a layer above the asbestos mat. The filtered solids are dehydrated with about 25 ml. of acetone, and then washed with tri-chlorethylene, benzene, or other solvent to remove adhering bitumen.

Calculations

The crucible and contents are then dried to constant weight in an oven at 105° C. The weight of material so recovered is expressed as a percentage of the weight of aggregate finer than the No. 100 sieve in the original 50 gram sample, and is designated as the "Loss in the Wash Test."

$$\% \text{ Loss in the Wash Test} = \frac{A}{B} \times 100$$

Where A = increase in weight of the gooch crucible, less the weight of the filter aid added.

B = weight of aggregate finer than the No. 100 sieve in the original sample of mixture, or

$$B = \frac{0.18 \times 50}{1 + \frac{\% \text{ oil}}{100}}$$

NOTES

1. This test is a modification of a method described by A. W. Dow in his paper, "The Wash Test in Differentiating Between Aggregates for Colprovia Paving Mixtures," Proceedings of the Association of Asphalt Paving Technologists, (1936).

2. The term "stripping" is used to describe two phenomena, one of which is the alteration of the oil film to a "dead," undesirable form which reduces or destroys the coherence of the mixture. The other consists of the actual removal of variable portions, but not all, of the oil film from the aggregate surface. Although the degree of "stripping" is measured in the fractions finer than the No. 100 sieve only, there is often substantial evidence that coarser particles have been affected similarly.

3. Aggregate graded up to ¼" is used to provide a mixture that approaches mixtures used in practice more nearly than would be obtained with finer aggregate. The presence of the coarser particles facilitates mixing and assists in breaking up agglomerations of mixture during agitation in water. Aggregate passing the No. 80 sieve and retained on the No. 100 sieve is omitted to facilitate a cleaner separation on the No. 100 sieve in the test. The percentage of aggregate finer than the No. 100 sieve should be determined by wet sieving.

4. The sample of aggregate used should weigh not less than 500 grams. If it is desired to make a swell test of the same mixture, 1,000 grams of aggregate should be used.

5. Hygroscopic moisture has a pronounced effect on the degree to which aggregates are "stripped" of their coatings. Warming to higher temperatures may cause a marked increase in adhesion. Shorter periods of heating at 140° F. may result in less adhesion.

6. While products as heavy as RC-4 can be mixed with considerable effort, the heavier products can be used more conveniently if cut back to a viscosity of about 400 seconds Saybolt Furol at 140° F. When testing cutback products, the procedure used to permit evaporation of the solvent should be standardized, since bituminous mixtures become more resistant to "stripping" as curing continues and the rate of change is accelerated with increase in temperature. If the materials under test are to be used in a hot plant mix, temperatures of warming should be increased accordingly.

An increase in the severity of the test for heavier products can be brought about by soaking the mixture in water a definite length of time before agitation.

7. A mixing container of low heat conductivity should be used to conserve the warmth of the materials until mixing is completed. This can be aided further by warming the earthenware bowl, or procelain mortar, in the 140° F. oven previous to mixing.

8. Average aggregates (Sp. Gr. 2.65) require about 5.3% of bituminous material by weight of the untreated aggregate.

9. Selection of a representative sample is facilitated by screening the mixture roughly through a No. 10 sieve and using proportional parts of the two fractions.

10. The sample for immediate testing (0 hour) is weighed out first, others are then weighed and placed in the 140° F. oven, and the time noted. These samples, after having been cured definite time intervals (4 hours is found to be convenient and occasionally 24 hours also, in extreme cases) are cooled for 20 minutes in the same manner as was the 0 hour test, before shaking with distilled water.

11. The spray nozzle used in connection with the determination of specific surface of cement by the Wagner Turbidimeter is convenient for washing the oil mixture.

12. It is important that the sieves be free from oily films which would prevent water from passing through readily. The sieves may be cleaned by washing first with solvent, then with a nearly boiling concentrated solution of tri-sodium phosphate, and then rinsing with water.

13. In most instances a total of 800 to 900 ml. of water will be collected. If the amount of "stripped" particles is exceptionally large more wash water will be required.

14. When the water is syphoned off, the beaker walls should be washed down with water immediately to prevent the retention there of any particles upon drying.

Without the use of filter aid the filtration may consume a long period of time, especially in the case of large amount of "stripped" fines. The filter aid so far used in this laboratory has been "Speed Plus" (The Dicalite Company) which has proved to be very satisfactory. Mixing of the filter aid and solids, which is essential, is accomplished by a jet of water from a wash bottle.

A Critical Discussion of the Present Status of Soil Mechanics as Related to the Successful Performance of Bituminous Surfaces

DEVELOPMENTS IN SOIL MECHANICS AS RELATED TO DESIGN AND CONSTRUCTION OF BITUMINOUS SURFACES

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The past ten or fifteen years has seen a remarkable development in a somewhat new field of engineering that has become generally known as soil mechanics. Many of the older engineers and road builders frequently point out that what is called soil mechanics today is not new but has been practiced since engineers first built roads, or structures, or anything which inevitably encountered soil as one of the materials to be controlled. In one sense it is true that there is nothing new about soil mechanics for it is a fact the problems that are the subject of so much interest today are the same old problems. However, it must be admitted that the method of attacking these problems certainly has changed radically and appears to be producing results which are much more satisfactory.

Progressive thinkers in this field are no longer satisfied with the old empirical methods and are not content to make their decisions purely on an individual's personal judgment, even though that judgment may have served its purpose very well in the light of generally inadequate knowledge of the subject. Fundamental research and the desire of an ever increasing number of engineers for more positive criteria to regulate the use of soil is not to be denied and should not be resisted by those who are not in a position to take part in or even follow the present development.

At the same time it must be admitted that "all is not gold that glitters" and there probably are a number of developments which are now regarded as important contributions that will not stand the test of further investigation and trial in actual practice. In this respect, however, the development of modern soil mechanics is no different than any other concerted effort which finally results in a definite accomplishment.

While it is perhaps too early to evaluate some methods and principles that are so new that they have hardly been put into practice, it may be beneficial to at least consider those problems in bituminous construction which may have profited from soil mechanics research.

A bituminous surface may be described in simplest terms as a structural element or slab resting on a soil foundation. Both of these component parts of the road have been subjected to intensive study originating to some extent from research in soil mechanics. because investigators in this field have been forced to consider all conditions associated with soil materials in order to describe their behavior in terms of fundamental physical properties.

Thus investigators in soil mechanics have been led into a study of the characteristics of the loads applied to such surfaces, the design of granular mixtures, and the effect of various cohesive agents in order to evalute the physical properties of these mixtures which control the structural behavior of the surface slab. Also the foundation on which the slab rests gives rise to a number of problems which are more specifically soil mechanics. Bearing capacity of both cohesive and granular subgrades have required extensive investigation. In fine textured soils moisture changes which upset the equilibrium of the soil-water system resulting in excessive volume changes and loss of stability have led into the province of soil physics. Frost action and the spring breakup have required special study in some parts of the country, and in all cases drainage and methods of preventing movement of capillary moisture have appeared as matters of primary importance.

It may now be of some value to review such accomplisments as have been realized and to call attention to those problems on which investigators may concentrate in order to produce results of most immediate benefit. Problems which have to do with the subgrade soil or foundation of the road surface will first be considered.

Knowledge of those soil characteristics which portray the equilibrium of the soil-water system is in a well advanced stage. These problems were among the first to be undertaken. Led by the Bureau of Public Roads a number of investigators have made important contributions including the early work of Eno,[1 2] the investigations of Terzaghi,[3] Hogentogler and his associates, [4 5] and the study of frost phenomena by Taber,[6] Burton,[7] Benkelman and Olmstead.[8]

Tests which evaluate the moisture holding capacity of soils have become more or less routine and include the Atterberg limits, more commonly known as the liquid limit, plastic limit, and plasticity index. Volume changes, due to varying moisture content, may be accurately evaluated by shrinkage factors such as the shrinkage limit, shrinkage ratio, lineal shrinkage and expansion, and volumetric change.

Frost phenomena may also be placed among those problems quite completely understood although remedies are still open to development. Whether one leans toward Benkelman and Olmstead's mechanism of a fluctuating frost line or Taber's constant frost line theory will determine the type and extent of corrective measures to be adopted. Both theories

lead to the conclusion that the intermediate silt textures with the detrimental combination of high capillarity and high permeability should be removed from the subgrade within depths of frost penetration. These textures cannot be drained under ordinary conditions and are the most serious source of frost heaving and subsequent spring breakup. The theories differ in their treatment of drainable soils of coarse texture, Taber holding that frost heaving should not occur in sands and gravel, while Benkelman and Olmstead demonstrate conditions under which these soils do heave. The latter would, therefore, require much more attention to lowering the water table below frost penetration in the coarser soils, particularly in pockets and non-uniform subgrades.

The behavior of capillary water in clay subgrades which introduces serious limitations on the performance of bituminous surfaces has been the subject of much investigation but the present status of our knowledge is far from satisfactory. The closely related properties of capillarity and permeability have not yet been logically correlated and the extent to which improvement can be effected by lowering the water table has not been definitely established. It is well known that moisture holding capacity changes considerably with the temperature, and Eno has measured the yearly cycle of moisture concentration in the subgrade in several instances. These scanty data only scratch the surface and similar information on all of the various soil types is an outstanding gap in our knowledge of subgrades. The importance of this factor can be appreciated when it is pointed out that moisture concentration in surface soils is the source of volumetric change and decreased bearing capacity of the subgrade even though frost action may not have been present to any degree.

This leads to a discussion of subgrade bearing capacity which is one of the most important of the present-day problems in soil mechanics. In a discussion of soil bearing capacity it is necessary to subdivide soils into two main types, granular and cohesive, as the essential factors of soil resistance are radically different in each case. In both cases methods of measurement have been developed and conceptions of behavior formulated. However, there are conflicting viewpoints on both theory and methods of measurement and only time and the analysis of experience under practical conditions will determine which views will prevail.

There have been two schools of thought on bearing capacity of cohesive material. One group holds that bearing capacity is controlled by shearing resistance of the soil and failure takes place due to plastic flow or lateral displacement when the yield value of the soil is exceeded by a load application. It is recognized that consolidation of the soil for any given load is inevitable but may be accomplished by preliminary consolidation of the subgrade or, in many cases, may be allowed to take place after the road is put in service. In the case of poorly consolidated and non-uniform materials settlement due to consolidation may be the limiting factor, but in any event conditions of equilibrium may be achieved without continuous progressive settlement. However, if the yield value of the soil in shear is exceeded plastic flow and continuous deformation leading to ultimate destruction of the surface may be anticipated. The writer definitely subscribes to this viewpoint and, in fact, has accepted it as the controlling consideration in his own studies of bearing capacity.[9]

The other approach to the problem under discussion has in the past emphasized consolidation to the exclusion of failure in shear, holding that the shearing stresses sufficient to produce rupture of the soil structure are out of the range of ordinary practice and ignoring plastic flow which may occur at much lower values of stress.[10] While this extreme position has been gradually abandoned in the past few years, the proponents of the consolidation theory are inclined to question whether plastic soils have a definite static shearing resistance and they point to the rather unsatisfactory status of shear tests to uphold their views.

That there are no generally accepted procedures for measuring shearing resistance of cohesive soils must be admitted, but at least some methods have been developed and used in practical application with definitely satisfactory results. Various shear testing procedures are now the subject of a comprehensive study by the committee on soil tests of the American Society for Testing Materials under the chairmanship of Professor Converse of the California Institute of Technology. This investigation has been extended to some twenty soil mechanics laboratories in the country and should do much to standardize shear test methods. The writer's own experience with a field penetration method, which has been correlated with laboratory shear tests and load bearing tests in the field, leads him to take an optimistic view toward this problem. These studies, extending over a period of more than ten years, have included the use of shear test results in the design of substructures in a number of instances, with results that justify confidence in the ultimate solution of this problem.

As has been indicated, the bearing capacity of granular materials is a different problem and methods of testing and fundamental conceptions of resistance have been formulated which again have led to conflicting viewpoints. Whether or not supporting capacity is a function of internal friction, as held in the long accepted conventional theories, or is simply internal stability due to arching action or mutual support of adjacent particles in the mass is a matter of current debate. That internal friction cannot suppply a positive criterion of failure has been pointed out by the writer in a report covering the results of several years of fundamental research conducted by the Michigan State Highway Department and the University of Michigan.[12] Naturally then the choice between the two theories has been made as far as he is concerned and will be maintained unless evidence to the contrary can be produced. The latter viewpoint is not generally accepted and only continued trial will determine its more general acceptance in the utilization of granular materials and mixtures.

The tests which appear to be essential in evaluating the properties of granular material are the stabilometer test and a test to measure resistance to shearing displacement. The stabilometer test first developed in this country was that of Stanton and Hveem[13] of the California State Highway Department and has been adopted by others, including the writer, with the introduction of one change, that of testing to failure under a constant lateral pressure. In this form the stabilometer test is no different than the triaxial compression test now being used in a number of laboratories. It measures directly the relationship between supporting capacity

of an element of mass and the lateral support furnished to the element. Regardless of whether this relationship is formulated in terms of internal friction or independent of it, the test will in all probability remain as one of the standard routine procedures.

Even if shearing resistance is not a controlling factor in internal stability of a granular mass, it is an essential element in ability of a granular mass or a bituminous surface to distribute vertical load. In the surface it appears more accurate to designate the action as punching shear. Tests now developed or in the process of development include the shear box with a single shearing plane developed at Harvard and M.I.T.[14] and a transverse shear test cylinder with a double shear plane, used by Berry[15] for fine sands in the early tests at Michigan. Both of these testing devices provide for the application of normal load. Powers[16] of the Arizona State Highway Department developed one for granular mixtures containing coarse aggregate which appears to be best adapted for ordinary mixtures. Skidmore[17] some years ago also used a double plane shear test for bituminous mixtures.

There appears to be a general recognition of the necessity of a standardized shear test on granular materials to supplement the stabilometer test, even though the information may be used in different connections when applied practically. The shearing resistance of granular materials is also included in the work of the A.S.T.M. committee and the results of their study should assist in standardization as present data on actual mixture is scanty indeed. Skidmore has given the results of a study of sheet asphalt mixtures, but aside from this there has been reported practically nothing which would establish shearing resistance values for use in design in connection with gravel base courses or other types of bituminous surfaces. It would appear that this property is one of the essential factors in the rational design of such surfaces and the lack of reliable design data or uniform test procedure should be corrected as soon as possible.

There are several recent developments in testing technique which have not received the attention they deserve, although the pioneering work has been done. Reference is made to the well-recognized necessity for evaluating the effect of rate of loading and dimensions of the specimen in measuring the strength of any material. Vokac[18] has made important contributions in this connection in the compression testing of asphalt mixtures. His methods proposed for evaluating both time effects and dimensional effects have been simplified to the point of being readily adaptable to routine testing. At the same time, this work brings out the necessity of testing identical samples at several rates of loading for each of several specimens with varying dimensions. As a result it is now recognized that reliable results from routine testing require the preparation of a larger number of identical specimens which in turn necessitates more accurate control of proportioning, mixing, and compacting. These requirements are being recognized and, to the writer's knowledge, several laboratories are being equipped with most adequate devices for preparation of samples and accurate control of the variables mentioned above. At first thought the larger number of tests required may seem forbidding, but testing apparatus with automatic control of

rate of load and automatic recording devices are now available and eliminate much laborious detail to compensate for the more rigorous requirements of testing procedure.

The various developments in soil mechanics which have been made by the concentrated study of a number of specific problems have finally been coordinated and combined in general methods of utilization or practical application. There are two subjects which come to mind in which the more or less scattered knowledge of soils has been brought together to form the basis of generalization which has particular application to bituminous surfaces. These are soil classification and a general design procedure for flexible surfaces.

Soil classification is the subject of a paper submitted to this conference by O. L. Stokstad, Soil Engineer for the Michigan State Highway Department, and need not be discussed here beyond pointing out soil classification as one of the most important objectives of soil mechanics in highway construction. In a review of the present status of the field it is encouraging to note that a system of classification which has accumulated a tremendous volume of data on soils, presented in the form of soil survey maps, has been adapted to highway design and construction. Further than that it has been found to be a reliable and practicable basis for identifying soil types and evaluating their influence to the point of justifying definite subgrade design recommendations which has unquestionably raised the standard of construction.

With respect to the rational design of flexible surfaces, it is also encouraging that progress in research has progressed to the point where at least two comprehensive methods of design have been proposed. [19]That there are differences in these methods is natural as their authors have selected somewhat different variables as the controlling factors in the problem. Their practicability depends upon whether the controlling factors may be reliably evaluated by available testing methods and finally upon whether or not they predict behavior under actual conditions accurately. Both methods are the subject of a lively interest on the part of highway engineers and they should be subjected to trial under practical conditions at every opportunity.

For this purpose the Michigan State Highway Department conducted an extensive series of load tests on a flexible bituminous surface over both clay and sand subgrades. It is hoped that this investigation may be completed within a few months and the results made available for early publication. The analysis of field tests has been partially completed and the related laboratory tests are in progress. The analysis of field tests is very promising and, in fact, appears to show a substantial verification of the formulae proposed for design. [20]Whether the laboratory tests will prove reliable as a measure of the resistance developed under field conditions is yet to be determined.

In conclusion it is the writer's opinion that the present status of research in soil mechanics as it affects bituminous construction is indicative of an extremely healthy state of affairs. While there are, of course, a number of important problems yet to be mastered there is a general recognition of the knowledge that is lacking and steps are being taken

to fill in the gaps. Most if not all state highway departments and other engineering organizations have already provided themselves with personnel and facilities for putting into practice the developments of soil mechanics research. Almost without exception the men who have been given this task are joining in the vigorous pursuit of more adequate knowledge which has characterized the rapid development of soil mechanics, and highway engineers may look forward to even more rapid progress in the next few years particularly in the practical applications so much neglected in the past.

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Progress in Research on the Improvement of Soil Properties by the Use of Chemical Admixtures

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A person traveling across the country by car or by railroad can observe from his window distinct changes in the color of the plowed or uncovered land as he passes. If he is going straight west from the central part of the east coast, the color of the soil will change in succession from reddish yellow to grey-brown, dark brown, black, and back to brown and grey-brown. If the curiosity of the traveler is excited sufficiently to stop and examine the soil more closely, he will find it in some places sandy and pervious, in others fat and tight, in still others loose and mealy; some soils will appear to be of uniform character, others will have gravel and rock fragments all through and imbedded in a uniform appearing soil matrix. Anyone, coming into even very superficial contact with soil, like our traveler, will easily convince himself that the term soil comprises a host of materials, differing from each other in many respects. He will understand that a great deal of observation, experimentation and coordination is necessary to gain and systematize the knowledge on soil properties, which is necessary for a scientific attempt to improve these properties by chemical means.

The study of soil and soil properties owes its start, and its present status, primarily to investigators who are interested in widely different uses of soils and who have been applying different methods in the practical and scientific attack of their specific problems. The farmer and the potter, the agronomist and the pedologist, the mineralogist, the crystallographer, the civil engineer, the physicist, the chemist, and members of other professions, have contributed to the spotted map of today's knowledge on soil materials. The soil scientist, who desires to regulate soil properties for purposes of highway construction, has to draw from all sources of soil knowledge if he is to achieve best results.

THE VARIABILITY OF SOIL COMPOSITION

Soil materials, as found in nature, may contain a few, or all, of the materials enumerated in the following: minerals, organic matter in all states of decomposition, inorganic and organic sols and gels, elec-

trolyte solutions, matter in the gaseous phase and water. There is also an infinite number of possible variations in quantity, quality and state of subdivision of these components. The physical and chemical character of a specific soil depends upon the properties of its constituents, but it is not a simple summarization of these properties. Even if all of the components are uniformly dispersed throughout the soil, the presence of each component is liable to influence the reaction of all of the others. If the soil possesses a stable secondary structure, the properties of the components are often greatly overshadowed by the structure. In other words, the properties of the materials responsible for this structure may greatly outweigh the properties of the other constituents. There is a difference in character and function between a pile of bricks, lumber, and tiles, and a building constructed thereof. Similarly, the properties of a soil depend upon the properties of its constituents and also upon their structural arrangement.

DETERMINATION OF THE SIZE OF SOIL CONSTITUENTS

The type and extent of a chemical reaction between two substances usually depends not only upon the character, but also on the concentration of these substances in the reaction vessel. In soils, reactions are generally confined to the surfaces of their constituents. Therefore, the amount of such surface is of similar importance as the concentration factor in liquid or gaseous systems. Methods are available for determining the amount of internal surface in a soil by means of gas adsorption. The method generally employed in highway laboratories is an indirect one, whereby the amount of surface in a soil is calculated from the size of its constituents. For the determination of the size composition of a soil, combined sieve and sedimentation analysis are commonly used. The evaluation of sedimentation experiments is based on Stokes' law, which states:

$$V = \frac{2}{9} n r^2 (d-d_1) g, \text{ where}$$

V = terminal velocity
n = viscosity of liquid
r = radius of particle
d = density of particle
d_1 = density of liquid
g = gravity constant

The application of this equation to the measurement of the size of dispersed particles presupposes:

(a) complete dispersion by the action of the dispersing agent and the stirrer,
(b) spherical particles,
(c) absence of any appreciable hydration.

Since the cements holding the individual soil particles together may be of widely differing chemical character, dispersion by a standard procedure, and involving a standard chemical as dispersing agent, do not always give the same completeness of dispersion with different soils. Differences in the ions adsorbed on the surface of soil particles may change considerably the degree of dispersion, as measured by the standard Bouyoucos Test. Data concerning this effect are given in Table 1.

It is well known that many of the finer soil components possess plate-like shapes but, fortunately, the smaller a suspended particle, the less influence does the shape exert on the results of sedimentation analysis.

Of important interest in the evaluation of the sedimentation analysis is the fact that most of the finer soil particles are hydrated to a greater or lesser extent.

If the particle under consideration possesses a hydration hull of thickness R, making the total active radius r+R, then, under the assumption that the density of the hull is practically equal to that of the liquid, we obtain:

$r = \frac{2}{9} n r^2 \left(\frac{r^4}{r+R}\right) (d-d_1) g$. If with Stokes' Law:

$r = A$, under consideration of the hydration we obtain:

$r = A\sqrt{\frac{r+R}{r}}$ as true radius.

With podsolic clay containing Na as exchange ion, the ratio $\frac{R+r}{r}$ is often larger than 4. Assuming this ratio as 4, the actual size of the particle is twice that determined by the application of the simple formula. The recognition of this fact does not decrease the practical value of the standard sedimentation analysis. The data obtained there-

TABLE 1

Mechanical Analysis of Soils as Affected by Exchange Ions

Soil	Adsorbed Ion	Coarse Sand	Fine Sand	Silt	Clay	Colloids	Passing No. 40 Sieve
Cecil	Natural	1	14	9	76	62	100
"	Na	1	12	9	78	70	100
"	Ca	1	13	16	70	54	100
"	K	1	13	10	76	65	100
"	H	1	12	10	77	68	100
"	Ba	1	13	14	72	61	100
"	Fe	1	12	11	76	Floccul.	100
"	Al	1	13	14	72	"	100
"	Mg	1	13	9	77	59	100
Hagerstown	Natural	3	14	50	33	15	100
"	Na	0	13	47	40	28	100
"	Ca	3	14	53	30	11	100
"	K	3	15	52	30	13	100
"	H	3	13	42	42	23	100
"	Ba	3	14	51	32	13	100
"	Fe	3	15	41	41	27	100
"	Al	0	12	45	43	29	100
"	Mg	0	12	57	31	30	100
Putnam	Natural	1	11	55	33	12	100
"	Na	1	10	42	47	30	100
"	Ca	1	10	59	30	13	100
"	K	1	11	58	30	12	100
"	H	0	9	60	31	18	100
"	Ba	0	9	65	26	15	100
"	Fe	0	13	52	35	25	100
"	Al	0	11	60	29	18	100
"	Mg	2	10	55	33	16	100

The soils were prepared by percolation with agueous chloride solutions of the bases. The excess salt was washed out with distilled water.

with are to be considered as measuring to a certain extent the activity of the surface of the soil constituents. As such, they are of considerable importance. This example shows that the type of our scientific tools often influences the results obtained therewith, and that a critical analysis of these tools is of great importance if one wants to know the value and meaning of the results obtained with their help.

FACTORS AFFECTING THE ACTIVITY OF MINERAL SURFACES

The type and extent of a chemical or physico-chemical reaction depends upon the character and concentration of the substances entering this reaction. Soils possessing different chemical and physical characteristics on account of difference in parent material, and type and duration of weathering, will also vary in their chemical and physical reactions with other substances. The most important substances with which the soil particles come into contact and reaction are water and electrolyte solutions. According to what has been stated before, the relationship of soil to water is governed by the amount and chemical character of the internal surface of the soil. This soil-water relationship, on the other hand, governs the engineering properties of soils, such as the water permeability, swelling, shrinkage, consistency change with varying moisture content, and cohesive and adhesive properties.

The soil constituents contributing most to the amount of internal surface in a soil are those of the clay and colloidal size. Therefore, it is important that their chemical character be known. This chemical character is usually expressed by the ratio of silica content to the content of sesquioxides of iron and aluminum, and also by the type and amount of exchangable cations in the exchange complex; an increase in cation adsorption capacity and also in water affinity goes hand in hand with increasing silica content. But there does not exist a proportionality between the silica content and the exchange capacity. The reason why silicon hydroxides tend to release the hydrogen ion rather easily (and consequently exhibit cation exchange capacity) and the iron and aluminum hydroxides rather release the hydroxyl ion (resulting in anion exchange capacity), has been explained twenty years ago by Kossel by means of electrostatic considerations. The reason why the cation exchange capacity is not proportional to the silica content is the fact that the silica is not uniformly distributed throughout the clay and colloidal particles, but that these particles represent layer lattices with some of the layers richer, others poorer, in silica. This has been shown with the help of X-ray analysis and other crystallographic tools. These tools promise to furnish the exact pictures of the chemical and physical structure of the finest soil particles. Unfortunately, the progress in this field is necessarily slow and for quite a while to come, we will have to content ourselves, for practical purposes, with the help that is furnished by the knowledge of the silica-sesquioxide ratio and the amount and type of adsorbed cations of the specific clay under consideration. These exchange ions determine the water-affinity of the soil. But, again, this water-affinity is not for all soils directly proportional to the number of ions present, even if they are of the same kind, but this affinity depends also upon the geometric and sterometric form of the surface with which the ions are connected. This fact, also, can be explained by electrostatic considerations. The treatment of its practical importance will be included in the following section.

Data showing the effect of the silica-sesquioxide ratio and of the exchangeable bases on the properties of clays are found in Table 2.

TABLE 2

Physico-chemical properties of some colloidal clays.

PROPERTY	TYPE OF H-COLLOID					
	Bentonite	Lufkin Clay	Wabash Clay	Putnam Clay	Susque-hanna Clay	Cecil Clay
SiO_2-R_2O_3 ratio	5.0	3.8	3.2	3.2	2.3	1.3
Exchange capacity, m.e./gm.	0.95	0.82	0.78	0.65	0.47	0.13
Swelling, *cc./gm.	2.20	1.18	0.94	0.81	0.57	0.05
Heat of wetting, Cal./gm.	(16.0)	15.0	13.9	13.8	11.7	5.9
Hygroscopicity, (30% H_2SO_4), per cent by weight	21.5	20.1	——	18.1	15.5	6.1
Swelling, cc. / Heat of wetting, Cal./gm.	(0.14)	0.08	0.07	0.06	0.05	0.01
Swelling, cc. / Exchange capacity, Cal./gm.	2.44	1.44	1.20	1.24	1.21	0.41
Heat of wetting / Exchange capacity	(16.8)	18.3	17.8	21.2	24.9	46.4

*Measured by the Winterkorn-Baver method.

FACTORS INFLUENCING THE BINDER QUALITIES OF SOILS

Cohesion is, usually, defined as an attraction between substances of the same character; adhesion, of that between substances of different character. From this definition, it appears that, on the one hand, cohesion of a soil system is a summation of adhesion phenomena and, on the other hand, that knowledge of the cohesive character of a soil permits certain conclusions as to its adhesive properties if used as a binder.

Soils are commonly divided into cohesive and cohesionless categories. The scientific justification for this is limited, as can be seen from the following discussion. Cohesionless soil materials are considered as being coarse, such as gravel, sand and silt. When moisture is introduced in a dry system of these materials, it is first spread over the entire available surface in a thickness which varies with the water affinity of the surfaces of the various components. The forces holding water on the above-named surfaces are relatively short ranged, and water in excess of that needed to build up this thickness will accumulate on the points where the mineral particles approach each other and in the lower part of the system. It will build bridges between the particles and hold them together by means of its surface tension. If the pores of such systems are entirely saturated with water, there are no air-water interfaces left and the systems lose their coherence. Accordingly, these materials show a curve with a maximum if cohesion is plotted against moisture content. This surface-tension kind

of cohesion is the only one present in all soil materials from gravel to clay, as long as these materials possess only short range adsorption forces. Of course, the amount of cohesion in such a system is a function of the number of connecting films present and of the radii of their curvature. Since these radii can become extremely small in the case of plate-shaped fine particles, great cohesive values may be obtained with materials possessing a large amount of such particles. Other factors also change with the shape of the particles, the most important of which is the maximum thickness of the adsorption film.

ELECTROSTATIC FACTORS OF IMPORTANCE IN ADHESION

From electrostatic considerations, it has been shown theoretically and on the basis of experimental evidence that the maximum thickness of the adsorbed water film on soil particles is a function not only of the charge, type and amount of exchangeable ions but also of the geometric structure of the mineral surface. The smallest thickness is obtained with a convex and nonuniformly charged surface, the largest, with a uniformly charged plane surface. In the first case, the hydration film is made up primarily of the hydratation water of the adsorbed ions: in the second, the ions swarm into the surrounding liquid, and the film includes not only the hydratation water but the free water in which these ions swim. This fact is of great importance for our problem insofar as systems of this kind of material may possess cohesion exceeding that of the pure water, even if entirely saturated with water. This is the case with bentonite. For plate-shaped particles, the following cases may be differentiated:

1. In the case of a very small electric surface field, and of a small range of surface forces, the cohesion is solely a function of the surface tension of water and the radii of curvature of the connecting water films.

2. Marked uniform charge with extended ionic atmosphere of equal sign tends to increase the thickness of the water film and causes repulsion of the particles. At very low moisture contents this decreases the cohesion, while at high moisture contents a pseudo-cohesion may be apparent because of the establishment of a certain structure (thixotropy).

3. A nonuniform charge with nonuniform ionic atmosphere can link two particles together by means of electric chains, starting at one point of one surface and leading over water dipoles and ions to an opposite charge of the opposite micelle. According to this reasoning, those clays which possess a plain structure with nonuniform surface charges, exhibit greatest cohesional properties.

ELECTROSTATIC ADHESION PHENOMENA IN MECHANICAL SOIL STABILIZATION

Data on the variation of the tensile strength of air-dry soil briquettes of equal clay content, indicate a maximum for soils in which the SiO_2/R_2O_3 ratio of the clay is around 3.2.

What has been said about the adhesion of particles of similar kind is, in principle, true where these particles come into contact with larger aggregates such as sand, gravel and crushed stone. For this case, the following conditions may exist.

1. Particles with uniform surface charge of the same kind exhibit no electrostatic cohesion.

2. Particles with uniform, but opposite, charges, and particles with nonuniform charges exhibit electrostatic cohesion.

These facts are exemplified by a phenomenon with which any practical road builder is familiar, namely, the better adhesion of podsolic clay to limestone and other basic aggregates, as compared with that to quartz and other acidic minerals. Of course, with the same amount of water in a system, the strength of the electrostatic linkage will increase with increasing polarizing power of the exchange ions and increasing dissociation from the surface.

These facts are of special importance in the construction of stabilized gravel (or crushed stone) road surfaces and bases. In these structures, consisting of graded mixtures of gravel, sand, silt and clay, the interlocking of the coarse material retains stability in wet condition, while adhesion of the fine particles produces cohesional stability in the system when dry.

There has been quite a discussion on what plastic index the soil fines should possess for optimum serviceability. In practice, a plastic index from zero to somewhat over ten has been found to give good results. Another question is whether this plastic index should be obtained by an addition of a small amount of clay with a high plastic index to the mixture, or of a larger amount of clay with a small plastic index. An answer to these questions must be based on the following observations:

1. An addition of fine materials to larger aggregates does not only fill the pores, but the fines are also adsorbed on the surface of the larger particles, thereby increasing the total volume. The thicker this film, the more the behavior of the wet system will approach that of wet soil fines. Therefore, only such an amount of fines shall be incorporated in the structure as will not appreciably separate the coarser particles from each other.

2. Soil particles with a high plastic index, i. e., with electrostatically active exchange ions, will improve the cohesion of the system if the aggregate is of an opposite electrostatic character. If it is of the same character, it will not only fail to improve the surface tension part of adhesion but may also cause decreasing stability of the structure in wet weather.

3. If a graded structure is used as a road base, its moisture content under natural conditions is usually sufficient to minimize the electrostatic attraction effect even though optimum conditions are present for detrimental swelling. Therefore, for base structures, the plastic index of the soil fines should be at, or close to, zero.

For surfaces, small amounts of clay with a large plastic index (with related marked electrostatic cohesion) and large swelling capacity appear preferable. In dry weather, the electrostatic attraction increases the stability of the road surface, while in wet weather, the swelling of the clay closes

the pores on the surface and prohibits the entrance and destructive effect of precipitation.

If hygroscopic substances are used for dust-proofing and for furnishing surface-tension cohesion on a stabilized surface, the same consideration should prevail as in the case of bases.

In general, a higher plastic index can be permitted in the case of an aggregate with opposite electrostatic properties from those of the clay than in the case of one possessing similar electrostatic character. If only a certain kind of clay is available, its character can profitably be changed by base exchange (Table 3).

TABLE 3

Change of Soil Properties by Variation of Exchange Ions

Change of Soil Property in Per Cent of Numerical Value for the Natural Soil

SOIL	H	Na	K	Mg	Ca	Ba	Al	Fe	Soil Property
Cecil—									
+9.7	+12.9		+4.8	–4.8	–12.9	–1.6	Flocculated		Colloid Content
–8.2	–18.9		+2.7	–8.2	–8.2	–2.7	–16.2	2.7	Plasticity Index
+1.33	+1.33		–1.3	+4.0	+1.33	+6.7	–8	+4.0	Liquid Limit
0	+2.4		0	+9.5	0	0	+4.8	+2.4	Centrifuge Moisture Equivalent
–2.4	+7.3		+2.4	+9.8	+7.3	+2.4	–4.9	+2.4	Field Moisture Equivalent
+3.26	+1.09		+1.09	+3.26	+1.09	+3.26	–3.26	+1.09	Speed of Compression (Function of Permeability)
Hagerstown—									
+53.3	+86.7		–13.3	–13.3	–26.7	–13.3	+93.3	+80	Colloid Content
–19.2	+53.8		–3.8	+3.8	–11.5	–19.2	–38.4	–38.4	Plasticity Index
–9.1	+29.5		0	–22.8	–6.8	–11.5	–9.1	–11.5	Liquid Limit
0	+248*		–10.3	+3.4	–10.3	+3.4	0	–3.4	Centrifuge Moisture Equivalent
+9.1	0		0	–4.5	–4.5	0	+9.1	+18.2	Field Moisture Equivalent
+38.3	–33.3		+11.7	+5	+8.3	+31.7	+53.4	+53.4	Speed of Compression (Function of Permeability)
Putnam—									
+50	+150		0	+33	+8.3	+25	+50	+108	Colloid Content
–5.9	+88		+5.9	+29.4	+23.5	+11.8	–11.8	–5.9	Plasticity Index
0	+37		+5.7	+11.4	+8.6	+5.7	+2.9	+2.9	Liquid Limit
0	+211*		–3.7	0	0	0	0	–7.4	Centrifuge Moisture Equivalent
+4.3	0		–8.7	–8.7	+4.3	+8.7	+4.3	+4.3	Field Moisture Equivalent
+25.3	–46.6		0	–4.0	+1.3	+12	+16	+21.3	Speed of Compression (Function of Permeability)
H	Na		K	Mg	Ca	Ba	Al	Fe	

* Waterlogged

THE ACTION OF SPECIFIC CEMENTING SUBSTANCES IN SOILS

Thus far, only that part of cohesion has been treated which is caused by the surface tension of water and by the linking effect of the water dipoles. In a number of soils, specific cements are active in effecting the adhesion or conglomeration of the ultimate soil constituents. The action of these cements may be compared to that of calcium carbonate or iron and aluminum compounds holding the sand grains together in a sand stone. Some of these cements are of inorganic, some of organic, nature; the action of some is easily reversible, the action of others almost irreversible. It is important, for intelligent soil stabilization, that the chemical character of these cements be recognized. Great advance is now being made in this respect by the use of microscopic methods, permitting the determination of the siliceous, ferruginous or aluminous character of these cements with the help of indicators. The importance of organic cements has also been recognized for a long time, but exact and easy tools for their investigation are not yet available.

It has been shown that knowledge of the chemical and surface-chemical properties of soil materials is of considerable help to understand the function of soil in mechanical stabilization. For chemical stabilization, this knowledge, by necessity, represents the starting point for all real advance.

EFFECT OF SALT TREATMENT IN CLAY — GRAVEL (CRUSHED STONE) STABILIZATION

In connection with mechanical soil stabilization, certain salts have been used to a considerable extent. The action of concentrated salt solutions on the properties of soils is not yet completely understood. But qualitatively, one can enumerate the following effects:

1. Adsorption of ionic constituents of the salts on the soil particles and changing of their surface-chemical character. This effect is specific for the different ions.

2. Increase of the surface tension of the soil moisture, and, therefore, greater surface-tension cohesion.

3. Equalization of moisture content by the deliquescent, hygroscopic and water-retaining properties of the salts.

4. Decrease of volume change in drying, owing to the crystallization of the salt.

5. Decrease of the swelling capacity of soils because of the presence of electrolytes in the soil moisture.

6. Increase of the solubility of components of the stabilized soil system in the electrolyte solution, as compared to that in pure water (limestone in NaCl solution); and possibility of recrystallization of this material with a specific cementing action.

At the present time, calcium chloride and sodium chloride appear to be used to the greatest extent. Others have come to be used extensively in certain locations. Certain of the effects enumerated above are specific for certain salts; others are general. Typical salt effects on soil properties are shown in Tables 4 and 5. At times, salts are added to soil aggregate mixtures for the purpose of chemical reactions with components of the

latter with ensuing production of special cementing substances. This is the case when solium silicate is added to calcareous mixtures—a procedure used to quite an extent in France.

TABLE 4

Intake of Electrolyte Solutions by Soils

SOIL	Ca-Putnam	Na-Hagerstown	
Conc. $CaCl_2$	Intake cc/g	Conc. NaCl	Intake cc/g
0 (H_2O)	.62	0 (H_2O)	.78
.72N	.63	.34 N	.71
1.44N	.63	.68 N	.72
2.9 N	.60	1.35 N	.68
5.8 N	.56	2.70 N	.69
11.5 N	.50	5.40 N	.65

TABLE 5

The Effect of Electrolyte Concentration on Consistency Indices of Putnam Soil

Salt	Milli-Equivalent of Salt Per gm. soil	Lower Liquid Limit	Lower Plastic Limit	Plastic Index	Volume Change	SHRINKAGE Limit	SHRINKAGE Ratio	Vacuum Moisture Equivalent
None	0	32.8	16.2	16.6	37.3	13.0	1.89	28
NaCl	.15	32.8	15.6	17.2	38.4	12.9	1.93	32.6
NaCl	.30	31.6	15.6	16.0	34.3	13.7	1.92	35.4
NaCl	.75	28.7	15.2	13.5	26.1	14.7	1.86	31.7
NaCl	1.50	25.8	12.7	13.1	11.9	18.9	1.72	32.3
K Cl	.15	31.9	21.1	10.8				23.6
K Cl	.30	30.8	21.0	9.8				26.3
K Cl	.75	30.8	21.4	9.4				25.4
K-Cl	1.50	27.7	20.3	7.4				23.2
$CaCl_2$	.15	30.9	14.3	16.6	34.6	12.9	1.92	26.1
$CaCl_2$	.30	31.7	15.3	16.4	36.8	12.8	1.95	25.9
$CaCl_2$	.75	30.7	13.7	17.0	36.5	12.2	1.97	25.1
$CaCl_2$	1.50	28.2	11.0	17.8	30.4	12.8	1.90	23.4

SURFACE-CHEMICAL FACTORS IN BITUMINOUS SOIL STABILIZATION

In bituminous soil stabilization, bitumen is added to soils for two general purposes. It is supposed to modify the water affinity of cohesive soils, while interfering as little as possible with their cohesive properties, and, on noncohesive soils, to introduce cohesion or to act as a cement. The former soils are unstable under wet, the latter under dry, conditions. If bitumen is used for cementing purposes in noncohesive soils, it should

adhere well to the mineral surfaces and its consistency should not show marked changes with temperature variations and with time of exposure to the weathering agents. These properties are identical with those desired in higher type bituminous constructions. The adhesion problem has been treated in greater detail at last year's conference and to a certain extent at this year's; but a few remarks on fundamentals may be permitted at this time. These are:

(a) The adherence of one substance to another, or the adsorption of one substance by another, can be due to forces of non-polar, polar and intermediate type.

(b) Polar adsorption is, in general, connected with larger amounts of energy than nonpolar adsorption and will take place preferentially if both types are possible.

(c) A special case of polar adsorption is the formation at the interface of water and oil insoluble compounds by constituents of the adsorbents and the adsorbates.

(d) With bitumens containing polar constituents, such as asphaltic acids, anhydrides and resins, and the minerals generally, consisting of ionic lattices with more or less marked polar character, polar adsorption should be expected to play a major role. This conclusion has been confirmed by the experimental results of a host of investigators.

(e) The adhesion of a bitumen to mineral surfaces can be changed with the help of inorganic and organic primers of a polar nature.

(f) Primers which develop, or induce the development of, cementing films appear to be preferable to those which develop lubricating films, if it is desired that the cement introduce as much cohesive stability as possible.

In the bituminous stabilization of cohesive soils, the bitumen has the job to waterproof, rather than to cement. The characteristics of such soil—water—bitumen systems have only recently found thorough experimental investigation. They are too complex to be described in detail at this place. But the importance of polar adsorption phenomena, connected with the polar bituminous constituents, the water dipoles and the surface structure and exchangeable ions of the minerals is already apparent and deserves special emphasis. Summarily, it might be stated that soils exhibiting low cation and high anion exchange capacity are more easily waterproofed than those with high cation exchange capacity. Furthermore, the higher the valence of the adsorbed cation, the better is its effect in bituminous soil stabilization. The monovalent ions, especially those of lithium and sodium, affect, detrimentally, the stabilizing action of bitumen. But this effect can be minimized by the use of a primer which resinifies in the presence of alkali. It also appears that stabilization with asphaltic bitumen is more affected by the surface properties of the minerals than stabilization with high temperature tar. The results of an extensive investigation of the factors of importance in bituminous soil stabilization, which was carried out under the joint auspices of the Bureau of Public Roads, the Missouri State Highway Department, the University of Missouri, and the Direccion de Vialidad of Argentina, will be available in the near future.

SURFACE CHEMICAL PHENOMENA IN STABILIZATION WITH PORTLAND CEMENT

Rather recently Portland cement has found a place in soil stabilization. The empirical basis for its use has been laid by an extensive laboratory investigation of the Portland Cement Association, which was strengthened by cooperative experimental jobs with several highway departments. Because of the complexity of soil-cement systems and the relative newness of this development, not much investigation has been done from the chemical point of view. Therefore, little can be said in this respect at the present time; but an investigation has been started to discover any possible effect of the chemical composition of the surfaces of the soil constituents on the quality of cement stabilization. Considering the chemical actions involved in the setting and hardening of cement, it would be very surprising if the chemical character of the soil had nothing to do with the effectiveness of cement treatment.

It would take too long to treat all the possible chemical and surface-chemical reactions between the different soils and the multivariant materials which are used at present in soil stabilization. But it is worthwhile to be on the lookout for such reactions in any case in which a foreign material is introduced in the soil for stabilization purposes. While the optimum conditions for such treatments will continue to be developed empirically, if the chemical side of the treatment is taken into account, it will be possible to arrive at a theoretical understanding and a practical standard of design in a much shorter time than without this guidance. On the other hand, care must be taken that any chemical theory, or hypothesis, used in the development of soil stabilization is based on facts and is submitted to testing by facts before any large-scale application. It should be emphasized that not a smattering of chemical knowledge but a well-founded chemical education and practical experience is necessary to do a good job in this field. As in other fields, half-knowledge is the worst enemy of advance.

In conclusion, it might be stated that chemical and surface-chemical phenomena play an important role in the different procedures of soil stabilization. The recognition of these phenomena and the application of the acquired knowledge is one of the foundations for the continuous and sound development of low-cost road construction.

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Progress in Research on the Adhesion Between Mineral Fractions and Bituminous Materials

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Last year, a report was submitted to the Montana National Bituminous Conference on the theoretical and practical status of the adhesion problem in bituminous road construction. (1) It is the purpose of the present paper to bring this information up to date. Also, since one year is a very small unit of time, and the progress made therein relatively limited, some space might be used for restatement of fundamentals; and for the enlarging on some features, which deserve more space than could be allotted last year.

FUNDAMENTALS

The theory of the adhesion interrelationship of mineral surfaces and bituminous materials is well advanced. It has been demonstrated that the resistance to stripping of bitumen coated aggregate is dependent upon:

(a) the surface-physical and chemical character of the mineral,

(b) the physical and chemical character of the bitumen,

(c) the time and condition of contact between the bitumen and the mineral.

According to their surface-chemical and physical properties, minerals may be divided into four classes containing:

(1) those minerals which possess a greater natural affinity for bitumen than for water (hydrophobic).

(2) those which possess a greater natural affinity for water than for bitumen, but the affinity of which can be altered beneficially (or detrimentally) by base exchange and other ionic adsorption phenomena. These minerals belong to the hydrophilic class but they deserve a separate denomination (perhaps amphophilic).

(3) those, like quartz, which do not possess base exchange capacity, and which, therefore, cannot be altered with the help of exchange ions (hydrophilic).

(4) those which possess a chemical nature similar to quartz; but which, because of a porous structure, permit the intrusion of biutminous plugs, resulting in a type of mechanical adhesion. If such aggregate is primed with salts of higher valent cations (Pb, Fe, Al), these may become retained in the pores, and change into insoluble oxy and hydroxyl compounds altering the affinity of the surface of the mineral for bitumen and water respectively.

The action of bituminous materials varies to a great extent with their chemical constituents. Unfortunately, at the present time there does not exist a good standard method to separate these constituents and to identify their chemical character. The Marcusson method which is the most complete one developed for the group analysis of asphaltic bitumen appears to give good results with materials for which it was originated, i. e., natural asphalts and noncracked and oil asphalts; but fails if called upon to chemically identify the multivarious cracked and processed materials which are now on the bituminous market. In regard to the adhesive properties of asphaltic bitumen it appears to be well established that best adhesion is obtained if the bitumen possesses a sufficient amount of active components which are preferentially adsorbed by the mineral and, possibly, form water and oil insoluble compounds with constituents of the mineral. These active components may be very stable, as are the acids found in many natural asphalts; or they may be very unstable if produced by excessive heating. Compounds showing special affinity for the hydrophilic, quartzlike, type of aggregate are usually of the latter type. While exhibiting desirable adhesive properties they, generally, possess an undesirable weathering susceptibilty. Therefore, bitumen containing this type of components has fallen into disfavor in the conventional semipermeable types of bituminous road construction. The greater stripping resistance of cracked bitumen has been demonstrated in three recent investigations (2, 3, 4). These investigations also confirmed the known fact of the influence of the consistency of bitumen on its stripping resistance. Films of low consistency and also those of a brittle nature are more easily stripped than plastic films of relatively high consistency.

In the light of present knowledge it appears most desirable to use a hydrophobic aggregate in connection with an asphaltic material with good weathering resistance. If a hydrophobic aggregate is not economically available the use of a more active bitumen may be considered; but if the greater activity has to be paid for with a decreased weathering resistance the wisdom of such procedure appears to be very doubtful. It would be better, in such a case, to attempt to change the surface character of the aggregate and to choose the bitumen on the basis of its weather resistant properties. The main problem in the field of adhesiveness then, is to change the surface properties of such hydrophilic mineral materials which are not susceptible to improvement by base exchange or which do not possess the above mentioned advantages of a porous structure. An attempt towards the solution of this problem is contained in one of the papers referred to previously (4). At this point the statement may be permitted that, while in most cases it is desirable to use a dense bituminous mix even with highly weathering resistant bitumens, this density is categorical for mixes containing a bitumen of low weathering resistance or a bitumen aggregate system with components having small mutual affinity.

THE TESTING FOR ADHESIVENESS OR RESISTANCE TO STRIPPING.

Two points deserve primary consideration in the actual testing of adhesiveness. They are:

(1) that the adhesion measured is a function of time and condition of contact of bitumen and mineral,

(2) that good wetting properties are not synonymous with good adhesion properties or good stripping resistance, though they may go parallel with the latter.

Accordingly, for the testing of stripping resistance such methods have to be ruled out which:

(a) do not allow sufficient time of contact before testing,
(b) measure wetting rather than adhesion characteristics.

It is not intended to minimize the great practical importance of wetting phenomena in the bituminous paving industry. But these belong under a different heading. A well known test which fails to take into account the point under (1) and other important factors is the water-asphalt preferential test. Although "de mortuis nil nisi bene" this test offers a classical example of misdirected testing methods and psychology, that it deserves closer consideration. The test was first developed in Arizona for a possible application in determining the suitability of materials for use with oil, especially materials passing the No. 200 mesh sieve. The original test procedure was as follows (5) "5 gm of that portion of material which is to be oiled is mixed with an excess of oil (27 ml) at a rate of 1500 revolutions per minute for five minutes; after which enough water to make up 80 ml is added, and the total mixture is then stirred for another five minutes. This procedure is carried on in a 100 ml flask at a temperature of 130° F.

"After the sample has had time to settle and the water to become clear the amount of clean dust at the bottom of the jar is estimated and recorded as the percentage of total dust separated."

(N. B. California doubled the quantities and raised the temperature to 140° F. (6).

On the basis of the general properties of bituminous oils, soil materials and water, a number of different phenomena can be visualized as a result of the testing procedure. Such possibilities are:

(1) the mineral goes to the water phase,
(2) the mineral goes to the oil phase,
(3) the system represents an oil in water emulsion,
(4) the system contains a water in oil emulsion in a surrounding water phase,
(5) the mineral is distributed between the water and oil phase,
(6) the mineral accumulates at the interface of the oil-water system.

Obviously, from the six enumerated possibilities, only three give more or less definite indications of the properties of the mineral. This is bad enough, but the actual value of the test is still more lowered on account of the fact that:

(a) the mineral has not been given sufficient time to react with the bitumen before testing.

(b) the mineral bitumen mixture has never been permitted to dry out and develop resistance against the stripping action of water. In the light of present day experience and understanding a wet laid bituminous road or base cannot be considered completed and possessing optimum water resistance as long as it has not lost most of its moisture (7).

The only definite conclusion to be derived from the use of the water-asphalt preferential test is, that a mineral which goes to the oil phase is a good material; but a mineral which does not do so is not necessarily bad. To reject materials on the basis of such a test is liable to become an expensive sport. It is interesting from a psychologic standpoint that the use of this test lingers on, in spite of the fact that its first investigators, the Arizona State Highway Department and that of California have early recognized and stated its limitations. A questionnaire submitted to the various State Highway Commissions during 1937 by a committee of the Association of the Asphalt Paving Technologists on the use of the water-asphalt preferential and the swell test resulted in replys from 23 states. (8). Of these, 15 used the swell test as criterion of water affinity, 10 used both the swell and the water-asphalt preferential test. Of the latter 10, six believed the swell test to be superior to the other, 3 were noncommittal, and only one voice appeared to favor the "preferential" test to the swell test. This voice belonged to a state, the specifications of which contain the swell test, not the water-asphalt preferential test.

THE SWELL TEST

This test also was originated in Arizona (9) for the purpose of determining the "suitability of a material for use with 65-70% asphalt fuel oil. In the testing procedure: "Material proposed for use with oil is mixed to satisfactory color—allowed to stand for 24 hours, after which that portion passing 10 mesh is screened out, tamped into a 2-inch section of 3-inch pipe and compressed to 200 lbs. per square inch against machined steel plate. After sealing the bottom with paraffin, the material and ring are immersed in water, After a twelve hour period, the swell above the top of the ring is recorded." A maximum permissible swell is specified.

This test suited originally, only for finer sized materials acquired more general importance in the able hands of Stanton and Hveem (6) in California. The improved "swell test is made on a compacted specimen of oil mixed aggregate representing typical proportions of oil or asphalt as well as grading used in actual construction." In the actual testing the mixture is compacted in a cylindrical metal mold of 4 inches inside diameter, and 5 inches high; then a perforated brass disc $\frac{1}{8}$ inch thick and $3\frac{7}{8}$ inches in diameter with a $5\frac{1}{2}$ inch upright stem in its center is put on top of the compacted mixture. The upright stem is in contact with an Ames dial, which indicates the swelling of the mixture, induced by 500 ml of water which are poured into the upper part of the mold on the surface of the specimen. Swell readings are taken after 24 hours and recorded in thousandths of an inch. The limits of the permissible swell are specified as functions of the relative surface area of the minerals in the mixture. Although one might well back up the statement of Stanton and Hveem that "the swell test is considered to

be the most reliable method and will throw more light on the probable effect of moisture on the road surface than any other known test'', it is well to examine more closely the inherent advantages and disadvantages of this method.

The advantages of the swell test are:

(1) the chance for the bitumen-mineral interface to develop stripping resistance.

(2) the possibility of testing the paving mixture in toto.

The main disadvantage is the fact that the measured swell is a complex function of the permeability and the water affinity of the mix. There exists the possibility that a highly hydrophilic filler in a semi-permeable pavement mixture may swell in a way as to close the pores to further entrance of water. Such a mixture may record a smaller swell than a less hydrophilic mixture but the corresponding pavement will not be able to withstand a long soaking water action as well as the one made of the other mixture. The test noting the 24 hours swell appears to be appropriate in places of excellent drainage but not in places of long continued exposure to moisture. To take care of this feature there is a tendency to increase the time of swell. This has been done in Nebraska where all tests are run for 140 hours. There it has been shown that often materials which will swell less than 0.015 in 24 hours will have more swell in 140 hours than mixtures having a swell of more than 0.015 in 24 hours. Another point which deserves consideration in this connection is that equal amounts of swelling do not necessarily result in equal decreases in stability. Where such decreases, due to the effect of water are of importance, it is well to know their magnitudes. Therefore the testing of stability of a compacted pavement mixture with and without prolonged exposure to water is well worthwhile. For the actual testing most of the known methods of the compression, extrusion, penetration, shearing and loading types appear applicable, as long as it is kept in mind that the specification limits must necessarily be correlated to actual road experience.

A testing method which is simple and well adapted to the needs of a country with high precipitation and slow evaporation of moisture is that descriped in the DIN 1995, Part II, Section D, Number 6 and 7. Cubes, made of the actual paving mixtures and compacted by a method which gives comparable results with the one used on the road, are stored for 24 hours in the thermostat at a temperature of 22.5° C., and then tested for their crushing strength. Identical samples are submerged in water for 28 days, kept still in water, in a thermostat at 22.5° C. for 24 hours, and are then tested for crushing resistance. Different limits are established for permitted decrease in crushing resistance of the samples. Typical limits are:

(1) Decrease permitted for ''Stampf asphalt'' ∠65%

(2) Decrease premitted for ''Guss asphalt'' ∠10%

(3) Decrease permitted for ''Walz asphalt'' ∠20%

Most engineers will concur on the point that tests on which the acceptance or rejection of a certain paving mixture is to be based should

be made on samples which are representative of the actual paving mixture. This fact limits the usefulness of the so called wash tests to pavements, the mineral components of which are of like surface-chemical character and of limited size variation, i. e., to some classes of natural and artificial rock asphalts, sheet asphalts and asphaltic macadam. The main application of these test methods is as research tools to be used in the furthering of our understanding of the factors of importance in the adhesiveness problem, in the development and testing of new priming substances and in the study of the functional relationship of the properties of the mineral components of a paving mixture and the properties of the mixture. At present almost nothing is known in the latter respect. Methods of the washing type are the shaking of the aggregate in a jar, the Nicholson (10) and the Missouri test (3,4). Nicholson's method, because of the use of Erlenmeyer flasks, is restricted to sand and filler material; the Missouri method can be adjusted for either large or fine aggregate. To be practically useful in their respective fields all these methods need to be standardized on the basis of actual road experience. Preliminary soaking in water or aqueous solutions is often necessary to make the results of these wash tests correspond with actual road behavior under many climatic conditions.

In the first part of this paper the factors determining the adhesiveness or stripping resistance of mineral-bitumen interfaces have been enumerated and methods to improve this quality have been indicated for all surfaces but those of a dense and hydrophilic character. In the coating of the latter progress has been made by the synthesis of very thin films of artificial resins on the mineral surfaces. Also, progress has been reported from the use of self-orienting polar molecules (Pb-soaps) at the mineral oil interphase (11,12). At this point it might be permissible to give a speculative picture of a hydrophilic mineral covered by bitumen and to derive a working hypotheses for the future attack of this problem.

A hydrophilic mineral is normally covered, even in apparent dry condition, by a thin moisture film. It takes very high drying temperatures to get rid of this moisture film, drying temperatures not feasible or economical in bituminous practice. Therefore, a hydrophilic mineral covered by a bituminous film represents a three-phase system consisting of the solid mineral, the moisture film and the bituminous film. The potential thickness of the moisture film depends upon the water attracting and orienting power of the mineral surface, the temperature of the system and the availability of water outside the system (vapor pressure). The actual thickness at any particular time is also a function of the resistance to the flow of moisture either in liquid or vapor condition through the bituminous film. If the water attraction of the mineral surfaces is equal to the resistance of the bituminous film to moisture flow and to its own deformation, no further change of film thickness will take place. Lowering the temperature increases the water attractive forces of the mineral, and, therefore, if other factors are kept constant, an increase in film thickness results. Under the influence of cooler temperature, the bitumen will get brittle and its film discontinuous. This will give a good chance for the under film to grow to appreciable thickness. Sudden temperature changes are able to change the adsorbed moisture into free water, which takes the space between the

mineral and the bituminous film and readily permits the stripping of the latter. Conditions like the ones described are set up in the winter-spring sequence.

The theoretical conclusions from this picture are that a good primer does not need to be entirely antagonistic to water. In fact, one can well imagine one which has smaller polar properties to water and will be adsorbed with like energy by the mineral surface. Its molecular weight should be considerably higher than that of water; but its molecules should have sufficient electrical likeness to those of water to permit a certain miscibility, which in customary fashion should decrease with decreasing temperature.

Such a primer would partly substitute and dissolve the adsorbed moisture film. Owing to its lower water solving power at low temperatures there would be no increase of the amount of moisture around the mineral and reduced danger of spring stripping; because of its higher molecular weight and its similar electric properties it would slowly substitute all the water molecules adsorbed on the mineral surface; and there would be a general tendency of the water to decrease on the surface of the mineral with increasing length of service of the pavement. Such a primer appears to be the ideal, and our search should be in that direction. Our experience with furfural seems to indicate that it possesses at least some of the desired properties.

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Progress in Research on the Testing of Bituminous Materials

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In spite of the vast amount of time and effort devoted to the development of tests for bituminous materials there remains much to be desired from the standpoint of correlation of test results with service behavior of the various types and grades of products.

Tests of consistency, such as the penetration test for semi-solids and the viscosity test for liquid products are known to be basic from the standpoint of suitability of the original material for a specific type of construction or method of use. However, numerous researches during the past decade have demonstrated that all bituminous materials, whether semi-solid or liquid, may be expected to change into hard, brittle solids with age when freely exposed in thin films to atmospheric conditions irrespective of increase in consistency due to evaporation of any volatile constituents which may be present. Some reliable test to evaluate the relative resistance of bituminous materials to undue hardening under service conditions is needed. Recent researches of Benson, Steinbaugh & Brown, Hubbard & Gollomb, Skidmore & Abson, Nicholson & Raschig and Doyle, all of which are descriped in the December, 1937 Proceedings of the Association of Asphalt Paving Technologists represent progress along this line which may ultimately lead to the desired test. The same may be said of the studies of Professor F. C. Lang, Minnesota Department of Highways, and of the recent investigations of the Materials and Research Department of the California Division of Highways.

With the knowledge that consistency changes begin to take place in all bituminous materials from their first application in highway use and that changes in consistency are accompanied by changes in almost every other physical property, it seems logical to assume that tests for other physical properties such as ductility at various temperatures, and softening point must be of minor importance unless the probable rate of consistency change can be evaluated.

It is quite possible to determine the relative tendency of bituminous materials to increase in consistency due to loss of volatile constituents. The present standard distillation test has been developed for this purpose and since the first series of conferences inaugurated in 1931 by the U. S. Bureau of Public Roads in cooperation with the various state highway departments and the Asphalt Industry, great progress has been made in the uniform classification of all liquid asphaltic road materials excluding emulsions, into three groups known as rapid curing, medium curing and slow curing, which are identified and specified by means of

this test. This basis of classification has resulted in a notable simplification of specifications for such products and the elimination of many test requirements formerly used which are now known to be unnecessary for specification purposes. While there is still room for improvement, the six tests commonly used for rapid curing and medium curing products, namely flash point, viscosity, distillation on the original material and penetration, ductility and solubility in CS_2 on the residue from distillation, have been generally found to be satisfactory. Unfortunately the same cannot be claimed for that class of products designated as slow curing which are produced from much more widely varying sources and by more widely varying methods. Tests which will more closely indicate their probable service behavior after use are still needed.

Tests for emulsified asphalts, first adopted by the American Society for Testing Materials in 1926, have as the result of subsequent research been revised five times since that date. Like many other tests which have been developed for bituminous material which is to serve only as one component of a finished product, there is need for direct correlation of the test to probable behavior of the material during and after combination with mineral aggregate components.

Probably the most important progress in the testing of bituminous materials has been made in the field of bituminous mixtures. As a result of numerous researches, engineers are showing increasing appreciation of the fact that the characteristics of mineral aggregates with which the bitumen is mixed or used are just as important from the standpoint of service results as the characteristics of the bitumen. They are also beginning to realize that interfacial relations between these two basic components which cannot in many cases be determined by testing them separately, is an extremely important consideration and that within rather close limits the proportions in which they are combined may mean the difference between success and failure under service conditions. It is believed that the day is not far distant when it will be generally recognized that tests on bituminous mixtures proposed for use are of far greater value in determining suitability and serviceability than are tests on the individual components before combination. When this realization is reduced to practice the problem of correlation of tests with service results will be greatly simplified.

Tests on bituminous mixtures may be conveniently divided into three classes: mechanical, physical and chemical. A brief review of the progress in research along these three lines may be of interest.

Mechanical tests are made on compacted bituminous mixtures and are designed to determine the resistance of such mixtures to applied loads. Various methods have been used to measure the resistance of these mixtures to deformation at maximum service temperatures for the purpose of determining the ability of the mixture to retain its original form in the pavement under the load and thrust of traffic. Stability, Compression and shear tests have been designed for this purpose and all have been found to develop information of practical value which could be correlated with service behavior. Researches along these lines by Skidmore, Vokac, Hveem, Hubbard and Field, and various other investigators have been published and are known to the majority of highway

materials engineers. Such a wide diversity of types of bituminous paving mixtures exist that no single test may prove suitable for all but there is need for standardization in this field so that resistance of a mixture to displacement under traffic can be expressed in terms readily understandable to the engineer.

Resistance of bituminous mixture to cracking under service conditions has generally been considered from the standpoint of the characteristics of the compacted mixture at low temperatures. The pioneer work of Rader in this field has pointed the way to securing much valuable data relative to the probable service behavior of bituminous mixtures in resisting stresses created by temperature and traffic which tend to produce cracks. Rader has utilized cross bending supplemented by impact tests in his researches, and has reported his findings in a number of technical publications readily available to the highway engineer. Raschig and Doyle following Rader have developed apparatus for a simple and convenient cross bending test and have confirmed Rader's conclusion that to resist cracking, high type asphalt mixtures should at low temperatures develop a relatively low modulus of elasticity accompanied by a relatively high modulus of rupture.

Physical tests of bituminous mixtures are also made on compacted mixtures. They include determination of density or per cent of voids and water absorption. The former are of especial value when taken in connection with the results of mechanical tests from the standpoint of mixture design or proportioning of the various components. Methods of void determination are simple and fairly well standardized insofar as determination of percentage is concerned. If, however, some practical method could be devised to determine the average size of voids in a compacted mineral aggregate the selection of the most suitable consistency of bitumen for that particular aggregate would be greatly simplified. This problem deserves serious consideration on the part of asphalt technologists who are particularly interested in low cost asphalt paving mixtures made with liquid bituminous materials as the bitumen carrying capacity of the aggregate is largely governed by both the percentage and size of voids in relation to the viscosity of the bituminous material.

Water absorption tests are often of direct value as related to service results, both in connection with possible injury of the pavement due to freezing and to interfacial reactions between the aggregate particles and the bitumen. This test while comparatively simple deserves further study and should be correlated with probable water absorption of the mixture as it exists in the pavement. It cannot be prophesied from a determination of voids alone, as some mixtures containing a rather high percentage of voids appear to seal themselves to water absorption at exposed surfaces when compacted.

What may be designated as chemical tests of bituminous mixtures are most commonly made on the mixture when uncompacted or in a loose state. The oldest of these is the determination of percentage of bitumen by means of solution in carbon disulphide or benzol. The methods in common use are reasonably satisfactory. However, methods of recovering the extracted bitumen in order to determine its individual properties need further study. The Abson and Bussow methods are suitable primarily

for solutions of bitumen obtained from mixtures containing asphalt cement. So far as the author is aware no acceptable method has as yet been devised for recovering liquid bituminous materials from mixtures, particularly when the bituminous material contains volatile constituents. The development of an accurate method for these materials would be of considerable use in connection with investigations of the changes occurring in bituminous materials when incorporated in mixtures subjected to actual service conditions or to accelerated weathering tests. The effect upon recovered bitumen of centrifuging the solution before recovery needs careful investigation as evidence exists that in some cases at least centrifuging throws out certain constituents of the bitumen so that the characteristics of the recovered portion do not truly represent the characteristics of the bitumen as it exists in the mixture.

Among the more recently developed tests for uncompacted bituminous mixture, those having to do with interfacial relations between the bitumen and the particles of mineral aggregate in the presence of water warrant special mention. In connection with bituminous surface treatments and use of open or semi-open type bituminous mixtures such tests are of great importance and are now being studied by a host of investigators. It is now generally realized that a substantial proportion of mineral aggregates are hydrophilic in character and that many failures of bituminous pavements under service conditions have been due to the use of such aggregates rather than to faulty characteristics of the bituminous materials. Nicholson was one of the first investigators to emphasize the importance of a water action or film stripping test for bituminous mixtures in general. Eberts and Powers had previously developed water action tests on mixtures of very finely divided mineral matter and bitumen designated as emulsifying tests and swell tests. These still are in service in evaluating certain bituminous mixtures but of necessity the field has been broadened to include all sizes of aggregates commonly used.

The Committee on Characteristics of Asphalts of the Highway Research Board conducted an interesting investigation of the resistance of bituminous materials to film stripping which was reported in the 1937 Proceedings of the Highway Research Board. A modification of the Nicholson test was used in this investigation by a rather large group of coworkers who concluded that the test was too severe for mixtures made with liquid bituminous materials. Further modifications appear necessary to make it generally applicable and there is much room for improvement in quantitatively estimating the degree of film stripping that may occur under the test.

Researches reported by Saville and Axon in the December, 1937 Proceedings of the Association of Asphalt Paving Technologists represent an excellent approach toward correlation of service results with film stripping tests. Two papers in the same proceedings, one by Winterkorn on "Testing the Adhesion Between Bitumens and Mineral Surfaces with Alkaline Solutions" and the other by McLeod on "Application of Surface Chemistry and Physics to Bituminous Mixtures" will be of interest to the technologist particularly concerned with this subject. It is of further interest to note that a subcommittee of the A. S. T. M. Standing Committee

on Road Materials has recently been created to develop a standard test for film stripping of bituminous mixtures under water action.

A number of the more recent investigations of resistance to weathering of bituminous materials mentioned in the first part of this report have been made on bituminous mixtures which have been subjected to weathering or service conditions. The bitumen has then been extracted and recovered to determine what changes in characteristics have been produced. While this appears to be a roundabout procedure it has certain advantages over attempts to determine changes due to weathering of the bitumen in mass. It appears quite possible that with further study changes in physical properties of bituminous mixtures under an accelerated weathering test may be made to yield as valuable information as any similar test on the original bitumen or the bitumen extracted from such mixtures.

It is realized that this report is by no means complete in covering all of the progress resulting in research on the testing of bituminous materials. It has been possible within practical limits of a paper of this sort to sketch in only a few of the high points. In conclusion, however, some mention should be made of the excellent approach to the development of tests for soil-bitumen mixtures in connection with the widespread soil stabilization program underway in many of our state highway departments. The work of McKesson in this field, as reported in the 1935 Proceedings of the Highway Research Board and that of Miller and Klinger as reported in the January, 1937 Proceedings of the A. A. P. T. are of very great interest. A number of tests and methods for evaluating such mixtures, described by these authors is extending our knowledge of how to measure characteristics which are considered desirable in soil asphalt mixtures.

Research Developments Related to the Service Behavior of Bituminous Materials

By

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and

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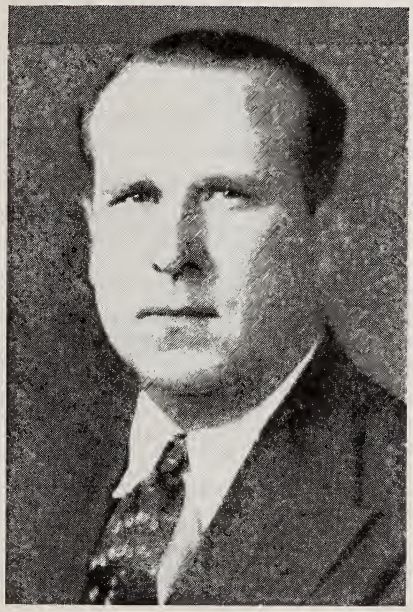

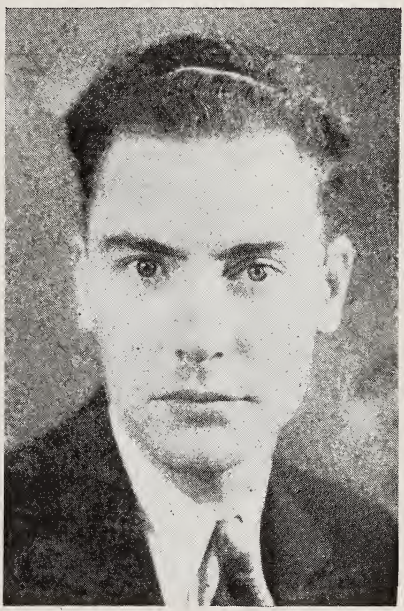

The "Service Behavior of Bituminous Materials" is a large subject and can cover most phases of research work on these materials. Any research has as its aim improving service, either by means of a new material, or by improving the property of an old one. However, we will confine our discussion to (a) information obtained by recovering bitumen from roads which have been laid for a period of time; (b) information obtained as to the mechanism involved in the breakdown of bituminous materials; (c) tests which have been developed to predict service behavior; and (d) how the engineer can possibly improve the service behavior of bituminous materials through the use of the above.

INFORMATION GAINED THROUGH RECOVERED ASPHALTS

We are all familiar with the fact that almost any paving material disintegrates either rapidly or slowly due to the influence of the elements. Rapid change in temperature tends to disrupt a pavement. The expansion of freezing water will break the bond of almost any binder. Surface water or ground water will often strip the aggregate of its

bitumen. And finally, light and oxygen will harden and change the character of the bitumen. Because of all these complicated reactions, many of the research technicians have thought the best way to attack the problem of "which bitumen is the best" is to extract the same from roads and streets which have been laid for a period of time and have given good or bad results.

Mr. Lloyd F. Rader[1] was one of the first men who pointed out, in conjunction with other tests, that paving asphalts continually lost penetration while in service. He showed that a Mexican asphalt of 42 penetration at 77° F. lost 29 points in about twelve years. He also recorded a Colombian asphalt of 56 penetration at 77° F. losing 22 points in nine years. Along with these penetration changes there occurred a rapid loss of ductility. The road surface became harder and more brittle. In fact, the asphalt which was only 13 penetration was cracking badly, even to the point of failure. Mr. Rader developed a series of low temperature tests on paving mixtures which he claimed would predict the performance of various mixes in the pavement.

Mr. Raschig and Doyle[2] of the Standard Oil Company (Ohio) followed up this idea and further demonstrated how asphalts hardened. In their investigation they showed that there was a great variance with the different asphalts used. For example, one street laid in 1928 with 48 penetration asphalt was still as high as 33 penetration at 77° F. in 1936, while another, laid during the same year, was only 12 penetration at 77° F. Another pavement dropped from 47 to 10 penetration at 77° F. in only six years. Although some of this hardening could be accounted for by overly hot mixers, etc., they showed that there was a wide variance in asphalt of different physical and chemical properties. These investigators were inclined to trust the "five hour loss in heating" test and "other cold temperature tests" for predicting the behavior of an asphalt in service.

Mr. J. W. Powers[3] reported before this conference last year on extracted bitumens in the state of Arizona. He found the same general loss in penetration and also a great variance in different asphalts. His report showed that any surface with asphaltic cement of 6 penetration at 77° F. or under was badly cracked. He reported the same loss in ductility in these hard cements, and a corresponding rise in softening point (R & B).

Mr. Powers[3] also investigated the loss in penetration by ordinary mixing in an asphalt plant at temperatures of 265° F. to 305° F. Here he states: "The penetrations of the extracted asphalt varied little if any from that which would normally be expected when mixes are kept in sealed containers." This is in contrast with the results reported by Mr. Tom Brannan[4], of the Ohio State Highway Testing Laboratory, who calls attention to "the high penetration drop in asphaltic cements, which average about 25% even when plant temperatures and mixing times are kept within the limits of the state specifications." This statement is further confirmed by Mr. Hubbard and Gallomb[15] in their studies of the hardening of asphalt in pavements. This difference may have been due to the difference in the asphaltic cement or to some difference in technique.

The only conclusion that all these investigators can agree upon is that the softer the asphalt, provided proper stability is obtained, the better and more serviceable is the pavement.

When road oil mixes are considered, a great amount of information concerning the recovered bitumen is available. Below is given a table showing some results obtained by the Arkansas State Highway Department. All recoveries were made by the Gene Abson method, and recovery was effected the same day as the extraction. The first eight samples were windrow samples. The other samples were dug from the pavement very shortly before the tests were made. The oil used was Oliensis Negative.

It will be noted that the first four samples had developed heterogenity and that the ductility dropped to a measurable quantity. All of the other recovered bitumens were Oliensis Negative and, with the exception of one sample of 6192, had 150+ ductility. The lower ductility of this one specimen was probably due to the large amount of mineral matter retained in the bitumen. Of interest also is the greater loss in penetration of the samples from Job No. 6189. This job was a retread open-type surface, while all others were dense graded mixtures.

Mr. Benson[26] before this body last year pointed out that tests on the recovered bitumen from road oil mats indicated that those asphalts having a high percentage soluble in carbon tetrachloride were producing surfaces of better quality than those materials having low solubility. He also stated that good ductility, when the road oil was reduced to a low penetration by the A. S. T. M. Method 243-28T, was a very desirable feature.

Mr. Zapata[5] noticed the great variation between "mats" identically made except for the source of the road oil. He therefore made a study of the constituents in the oil at the end of various periods of time after the "mat" had been laid. He reports the "hard asphalts" portion increasing from 15.4% when laid to 19.6% after 18 weeks.

Also Mr. Raschig and Doyle report that the general hardening takes place in other types of asphaltic surfacing materials besides those used in sheet asphalt and asphaltic concrete.

TESTS ON RECOVERED ASPHALT

Year Laid	Job No.	Sta. No.	% Water	% Solvent	% A. C.	Distillate in Bit. Part	Penetration	Ductility	% Ash	Oliensis	Type	Sample No.	Penetion	Ductility	Oliensis
1937-38	1165	162+60	4.08	.39	3.54	9.92	148	106	1.07	p	RC-2C	A-1715	140	100+	n
1937-38	1165	136+50	3.50	.31	3.74	7.65	134	115	1.38	p	RC-2C	A-1749	129	100+	n
1937-38	1165	856+00	.53	.24	4.10	5.53	102	100	3.16	p	RC-2C	A-1491	120	100+	n
1937-38	1165	953+00	.82	.21	5.11	3.95	129	107	3.08	p	RC-2C	A-1490	122	100+	n
1937-38	7241	234+00	2.32	.26	6.60	3.68	101	150+	1.19	n	RC-2C	A-1886	138	100+	n
1937-38	7236	242+00	3.05	.09	3.36	2.61	98	150+	1.25	n	RC-2C	A-1716	139	100+	n
1937-38	7220	35+00	.31	.05	4.77	1.03	73	150+	1.63	n	RC-3	A-1450	95	60+	n
1937-38	7220	26+50	.60	.04	3.39	1.17	79	150+	1.37	n	RC-3	A-1447	94	60+	n
1936-	6189	190+00	1.73	.15	5.25	2.78	62	150+	.23	n	RC-3	A-512	105	60+	n
1936	6189	300+00	.86	.12	2.92	3.93	48	150+	.46	n	RC-3	A-521	115	60+	n
1936	6192	136+00	.58	.19	2.39	7.37	91	116	7.44	n	MC-3	A-449	193	60+	n
1936	6192	Govt. Park	1.08	.41	3.94	9.43	120	150+	6.77	n	MC-3	A-440) A-452	198) 190)	60+ 60+	n n

Recoveries made in February, 1938.
All jobs were dense graded except 6189, which was a retread surface.
(Compliments of Mr. Wales and Mr. Roark—Arkansas Highway Department.)

PHYSICAL AND CHEMICAL MECHANISM INVOLVED IN THE BREAKDOWN OF BITUMINOUS MATERIALS

From the work cited above, it is established that the bitumen in a road surface does harden and lose its life, causing failure of the road surface. Let us now see what recent investigation has provided in the way of an understanding of mechanism of this break down.

Mr. E. O. Rhodes and H. E. Gillander[6] have given us a rather complete picture of the way in which tar breaks down. By controlling the conditions so that evaporation, oxidation, polymerization, effect of light and water could be studied separately or in unison, they were able to determine the effect of each and its magnitude. Their results confirm the work done by Sabron and Renaudie[7] that evaporation is the most active reaction and then oxidation and polymerization follow. Light has only a very small part to play, and water almost none. With this in mind it can easily be seen that a dense mix is imperative, and the life of the surface is governed by this feature.

The decomposition of asphalts is not so easy, and therefore some understanding of its make-up must be had. Asphalts (8, 9, 10, 11, 12) Bibl. are considered to be composed of asphaltenes, asphaltic resins, and oily constituents. Nellensteyn[11] and others consider that these constituents are combined as a protected lyophobe sal with the oily constituent as the medium and the asphaltenes as the dispensed lyophobe portion protected and possibly peptized by the asphaltic resins. That is, asphalts are made up of a colloid, mainly carbon, held in suspension and protected by resins. These resins slowly grade into oils, and are in solution in oils.

In many asphalts the colloidal dispension is of such a nature that time hardening occurs. Mr. Traxler and Combs[8] have developed an asphalt aging index and have developed a test which will show visually the thixotropic nature of certain asphalts. The significance of age hardening to service behavior of paving cements is not entirely known. It is known that highly blown asphalts show the most time hardening and some of the best paving cements show almost none. This test will be discussed later.

Not only does the colloidal structure of asphalt show time hardening, but Mr. Benson[13] has pointed out that upon microscopic examination certain films of bitumen which have been exposed to heat show a definite coagulation and finally due to lack of homogeneity. The reaction of the coagulated portion of the film to solvents might indicate that Mr. Traxler and Mr. Benson are attacking the same problem from a different angle.

Besides a rearrangement of the different constituents of asphalt, each one of the constituents are affected differently by oxygen and light. Mr. Thurston and Knowles[14] have shown that the asphaltenes are the most easily oxidized and that this oxidation is an absorption of oxygen. The oils are the least easily oxidized and lose weight when this occurs. The resins are between these two extremes. Of course the asphaltenes are supposed to be protected from the weather by the petro-

leum resins and the oils so that the rapid oxidation of the asphaltenes may not be so serious a matter.

The effect of light upon asphalts has been studied by many persons. Mr. Strieter[16] has developed a weathering test for roofing materials, in which the main factor is an ultra violet light. He has found that asphalt develops a decomposition product which is soluble in water and is in a large measure responsible for the failure of asphalt coating. Tests carried on with paving materials are not as conclusive, but it is recognized that light does have a definite part to play in the decomposition of asphalt.

Now, if an asphalt can fail in all of these different ways, how does it manage to last any length of time on the road? Do actual exposure tests confirm these reactions. Mr. R. H. Lewis[17] has made actual exposures of films to the weather, and has found "hardening . . . that may be attributed to oxidization, polymerization, and carbonization". Mr. Benson[13] states: "The results of natural weathering agree, in general, with the results obtained by other tests, although differences in time exist". It is fortunate for the road builder that, although these reactions described above do go on, they are very slow in at least some of the asphalts.

Now let us consider the tests which have been developed to exclude those less desirable asphalts:

A CRITICAL DISCUSSION OF TESTS DEVELOPED TO PREDICT SERVICE BEHAVIOR

1. Adhesion tests of different kinds are being proposed. These are of the most importance to the highway engineer, however, since this subject is being discussed in another paper, we will leave it without further comment.

2. The Oliensis[18] spot test detects a heterogeneity of a certain type It does not necessarily separate the bitumen which will give good service from that which will give poor service, but it does enable the engineer to tell the "state that is introduced into a petroleum asphalt through subjecting it during the refining operation to temperatures higher than is normal in the ordinary steam refining or vacuum process". To this extent it is an excellent test and deserves its wide popularity.

3. The "Fluidity Factor" introduced by Zapata[19] is a test which has been introduced as determining the source of the asphalt the same as the V. I. could determine the source of lubricating oils before solvent treatment was developed. The test is based on the idea that certain viscosity characteristics can be obtained by one and only one combination of the constituents of the asphalt. However, this is not true. In asphalts, the same as in lubricating oils, a low rate of change of viscosity with temperature change can be obtained either by the use of highly saturaated non-polar compounds, or by the presence of colloidal matter. In the case of Mexican asphalt the low rate of change of viscosity is due to a large amount of colloidal matter. In another bitumen from a mid-continent source, which has a high fluidity factor, the insolubility in 86° Be Naphtha is very low but the lubricating oil fraction obtained from the crude from which this oil is made has a high V. I.

This general contention is borne out by a simple experiment performed by the authors. A base flux of good ductility but low fluidity factor was obtained. To two portions of this flux two different oils were added in such quantity that an SC-6 road oil was obtained. The first was a high V. I. Pennsylvania oil. The other was a low V. I. oil. These two samples were then heated according to the A. S. T. M. test for residue at specified penetration. The results are shown below:

Original flux:

Float at 122° F.	281 sec.
Insol. in 86° Be naphtha	9.2%

Low V. I. Oil:

Flash, Cleveland O. C.	345° F.
Vis. Saybolt-Univ./100° F	138 sec.
Vis. Saybolt-Univ./210° F.	41
Viscosity Index	56

High V. I. Oil:

Flash, Cleveland O.C.	425° F.
Vis. Saybolt Univ./100° F.	199 sec.
Vis. Saybolt Univ. 210° F.	47
Viscosity Index	106

	% Fluxing Oil	% Flux	Furol Vis. @ 140° F. of Road Oil (Sec)	% Res. 100 pen. Av. 2 tests	Pen. @ 77° F. 5 of res.	Duct. 39.2° F. cm/min. (cm)	Vis. Furol 275° F res.	Wis. Fluidity Factor
Blend with High V.I. Oil	30	70	490	79.0	100	6¾	231	131
Blend with Low V.I. Oil	30	70	449	76.3	99	7	210	110

Now it may be asked of what importance is this as far as the service behavior of the asphalts is concerned. In lubricating oil, provided no additive compounds are used, it is generally conceded that the more the low V.I. products, or resins, are removed, the lower is its film strength. That is to say, the higher the V.I. of the oil is, the less does that oil tend to adhere to another surface. This is exactly opposite of what we want in a paving material. It may be quite true that an asphalt with a certain viscosity is needed in order to obtain the desired permanent film thickness on the aggregate, but it is not desirable to obtain this viscosity by the use of oils with low adhesions.

Although it is too early to evaluate the quality of the roads it is true that the roads built with the high fluidity, low insolubility in 86°Be naphtha, have given different results from the other type of oil.

So let us conclude that "Fluidity factor" by itself does not insure a bituminous material of a certain type. If it were combined with some test like the insolubility in 86°Be naphtha it might.

4. Another specification which is intended to insure quality is the "sulfur clause." Mr. Nicholson[21] has pointed out "that sulfur seems to have a great influence on the viscosity properties of the asphalts." Also

Mack[10] has pointed out that certain sulfur compounds seem to aid in the dispersion of asphaltenes. However, there must be much more research done before the role of sulfur in asphalt is understood.

5. A fifth test which has attained great popularity is the low temperature ductilities. The thought is that the asphalt must be ductile at low temperatures because of the great contraction and expansion to which the pavement is subjected in cold weather. A valuable addition to this test is the ductility tension test developed by Mr. Grant and Mr. Pullar[22]. The idea behind this test is, as Professor Neumann[23] has pointed out, that a certain strength is necessary in bituminous materials. Also, it appears as though a characteristic curve for an asphalt of definite composition could be developed if the specimen is tested at a lower temperature than 77° F. This test might give the engineer a quick, simple way of obtaining an idea of the percentage of resins, percentage of oils, and percentage of asphaltenes.

6. As has been previously mentioned, there are two tests which have been developed recently to determine the probability of the bitumen remaining homogeneous after it has been in service. One of these is Mr. Benson's[13] coagulation test. This test does appear to have great merit and does seem to be correlated with service behavior. The other test is the etching of bituminous surfaces as developed by Mr. Traxler. It is a test which shows the amount of time hardening which will occur due to the rearrangement of the colloidal structure of the asphalt. The results of this test may be correlated with those obtained with Benson's test. The exact field and function of this test has not been determined but should be given serious consideration.

7. Finally, Mr. Nicholson[24] has developed one and Mr. Raschig and Doyle another, both very practical, oxidation tests for determining what the penetration drop is of different asphalts during the time of mixing in hot mixes. This test should be of great service in selecting an asphalt which will not harden excessively during this operation.

Now in addition to the tests which have been discussed, it is hoped that some accurate and easy tests will be developed to evaluate the properties of the various constituents of asphalt, especially the resins. For if the resins extracted by the Furfurol and Duosol processes are representative, it is the resins which have the binding power and the only constituent that, by themselves, can bind the aggregate together to form a road. There should be tests to evaluate both the quantity and quality of these constituents.

THE IMPROVEMENT OF SERVICE BEHAVIOR

In summarizing this discussion, the ways of obtaining better service behavior will be discussed.

We have seen that there is considerable hardening during preparation of bituminous mixes. To reduce this hardening to a minimum, an asphalt should be selected which shows the lowest rate of hardening by an oxidation test, provided it has other good characteristics. Then, since

the amount of hardening is directly proportional to the temperature of mixing and the length of time the material is mixed, both should be kept to an absolute minimum.

The judging of an asphalt which has the proper constituents is much more difficult. It is known that if the oily constituents are increased so that they are in excess of 50% of the total bitumen in a 50 penetration asphalt, a product is obtained which has low ductility, low susceptibility to temperature, low tensile strength, and a tendency to rapid time hardening. On the other hand, if the resins are increased we get great susceptibility to temperature change but good ductility. The asphaltenes in themselves are a powder, but when added to the resins they decrease temperature susceptibility, add strength, and increase viscosity. Now, there is a proper proportion which will give the desired material. The engineer or chemist has to decide what proportions are necessary. He then can obtain the material by a "source" specification which may not contain any tests for the desired qualities, or he may write a quality specification only. The chemist is just now beginning to provide the necessary information to do the latter, and as yet there is almost no one who knows just how to apply the information gained.

From a refiner's viewpoint it is somewhat easier. After the engineer has written his specification he can blend products from the same crude or different crudes to obtain the desired results, as has been pointed out so ably by Mr. Holmes and Mr. Raphael[20].

In addition to obtaining the proper percentage of the various constituents, it is necessary to be sure that the constituents are of the proper character. This is where Mr. Benson's test, the Oliensis spot test, and others should be used.

In closing, let it be said that as yet nothing has been found which can be substituted for the experience and good judgment of the engineer. July 20, 1938.

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Progress in Research as Applied to Maintenance of Bituminous Surfacing

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A study of the inter-relation between research and maintenance of bituminous surfaces, as implied in the title of this paper, seems at first to lead to the conclusion that, strictly speaking, organized research is not often aimed directly at solving the problems of the maintenance man. That is to say, methods and technique utilized by maintenance departments for the purpose of correcting existing faults or damage to bituminous surfaces have not often been made the subject of research as the term is applied in the laboratory.

For the most part, maintenance methods and operations are the result of experience gained through trial and error methods. This is in no sense a disparagement of such procedures. It may be said of the maintenance man, as it has been said of his mother, his "work is never done." Confronted with unlimited places which loudly demand attention and money, and with limited funds, the maintenance man in every locality is an excellent example of a busy individual confronted with hard facts and stark realities, and it is not surprising if he seems to have little time for theoretical considerations.

However, a broader consideration of the application of research to maintenance indicates that in the end almost the sole purpose of research is to reduce the cost of maintenance. Even though some work is done primarily to discover cheaper ways of achieving satisfactory results, nevertheless the term "satisfactory" implies always durability and minimum maintenance expense, so that regardless of the particular problem investigated, the maintenance department is generally the ultimate beneficiary.

If highway organizations are viewed in a narrow, somewhat subjective sense as a group of separate departments or individuals invested with separate responsibilities and interests, the maintenance man inherits the benefits, if any, from research work; whereas the construction man frequently bears the burden of applying new ideas, and to him the researcher is mostly a nuisance. Actually, of course, work of the separate

departments does not usually have a clean-cut separation. Both construction and maintenance departments do occasional experimental and most maintenance departments do considerable construction work, but the net results are much the same, namely, that innovations may add to the expense and difficulty during construction. Therefore, for a given type of construction the justification for added expense can only lie in a commensurate reduction in maintenance costs. This, of course, does not imply that more expensive types of construction need be used. It is only meant to consider the improvement of the particular type which may or may not involve some additional expense.

In other words, if certain new procedures or processes are actually an improvement, the improvement can usually b eguaged only in terms of better service, and if sufficient saving in maintenance cost is possible, an increased first cost might be economically sound. Of course, really effective research should reduce both construction and maintenance costs, and make everybody happy.

It seems that a discussion of the effect of research on maintenance can rightfully include virtually all research related to the construction of highways.

Taking up specific cases and application by dividing the work under separate headings, first and inescapably comes:

(1) **Subgrade Difficulties.** So much has been said and written about the need for adequate foundations that one only states the obvious in stressing the need for good subgrades. In passing, it is interesting to note that while there are only a few subjects on which highway engineers are in anything like universal agreement, nevertheless there is no argument on this point. Sad to relate, however, many roads are still being built on foundations and subgrades that leave much to be desired.

Research aimed at improvement of subgrade conditions has been carried on for a number of years and is still energetically under way in many organizations. The first and simplest improvement usually consists of adequate drainage accomplished by deepening side ditches and so on. While raised grade lines or deepened ditches are frequently effective, there is a tendency toward increased traffic hazard, and if deep ditches are built with gentle shoulder slopes, costs often become a serious item, and at best, deep side ditches are frequently inadequate to remove water from heavy soils.

The assumption that ground water will always obligingly run down hill and drain from all manner of materials by gravitational flow is distinctly at variance with the facts. Gravel filled drains either with or without tile or perforated metal pipe have been used with some success, but are extremely liable to become inoperative due to silting up and are likewise often ineffective because capillary water does not choose to move exclusively down grade.

Therefore, another line of attack has been developed, namely the stabilizing of adverse soils so that they will not be too readily affected by water. The term "soil stabilization" has become about as all-inclusive and generally vague as the term "good engineering practice." It ap-

parently may be used to express almost anything that the speaker or writer has in mind.

Nevertheless, the findings of the researcher in soil mechanics only confirm what rule of thumb practice and common sense have long recognized, namely, that plastic, lubricated clay and heavy soils are improved by the addition of granular materials which increase the internal friction, and that loose, cohesionless sands can be improved by the addition of proper quantities of clay to supply the cohesion which is lacking. This, in its simplest form, is soil stabilization by judicious combinations of existing natural soil, sand, gravel, or stone.

Another attack on the problem, which is being largely investigated and developed by organizations seeking outlets for commercial products, involves the addition of certain artificial ingredients such as road oils, bituminous emulsions, portland cement, and sodium and calcium salts. The intent in each case is to preserve some desirable state of equilibrium. In the case of bituminous materials and portland cement, the chief purpose is to provide a water-resistant mixture that will not soften or lose supporting power when wet. Use of the various hygroscopic salts is based on the observation that most soils are satisfactory with a certain optimum moisture content. While California experience is limited, it appears that this treatment would be most successful when applied to a stable soil, one which did not become readily lubricated.

On the other hand, if an existing soil can be economically treated so that it will not absorb excess water, it may be expected to retain the supporting power which is characteristic of that soil in the comparatively dry state. Even the most adverse soils will usually support a pavement satisfactorily as long as the moisture content is below a certain maximum.

Failures of pavement surfaces due to lack of subgrade support undoubtedly represent the largest number of road surface failures chargeable to any one cause. This lack of support is practically always due to two factors—one, the existance of a soil which is readily lubricated by water; second, the presence of moisture to provide the lubrication. Moisture may enter the subgrade either by capillary action, water being drawn up through the soil by a force which is commonly termed "capillarity" (which is a misnomer in that the source of the attraction does not rest in the soil pores but rather on the surfaces of the particles themselves) or water may enter the subgrade through a porous pavement, and examples of failures from this cause are numerous. It is probably true, however, that more foundation trouble results from ground water than from water entering through the pavement, as it is usually possible to seal the pavement successfully.

The placing of seal coats to prevent the entrance of rain and snow water may at times defeat its own purpose, as it has been pointed out elsewhere[1] that many road surfaces of local material or water-bound gravel have reached a satisfactory equilibrium in which the moisture ris-

1. F. N. Hveem, "Some Basic Factors and Their Effect on the Design of Bituminous Mixtures," Proceedings of the Montana National Bituminous Conference, 1937.

Milton Hersey Company, Ltd., Montreal, "Notes on the Design and Construction of Bituminous Pavement Mixtures," page 60.

ing from the subsoil is dissipated by evaporation at a sufficient rate to prevent undue accumulation. If this same road surface is covered with a tight pavement or seal, evaporation will be stopped and moisture accumulated both by saturation of the pores from capillary moisture and from condensation of the vapor in the upper layers. It should be realized that the penetrating capacity of water in the vapor state is much greater than the liquid, and attempts to interrupt capillarity by placing open gravel with large pores will not obstruct the passage of water vapor, which may condense above the open layer.

If there is nothing above the granular strata which is susceptible to damage, no harm will result. However there is good reason to believe that many failures of dense graded oil mix type of surfacing are directly due to water vapor rather than to capillary water.

Laboratory experiments have been made in which dense graded bituminous mixtures were compacted to the equivalent of pavement density in the form of briquettes 2″ deep in a cylindrical steel mold 4″ in diameter. Specimens were prepared in duplicate, one sample being covered with a seal coat, and the companion specimen left untreated. All samples were placed in saturated sand and exposed to the sun for a period of one month. At the end of that period all samples were tested for stability. In every case, the samples covered by the seal coat were less stable than those without seal. The difference in stability was not great in the case of a well designed mixture, but was very great in the samples known to contain high percentages of clay, and adverse material.

One interesting fact brought out was that even though the sealed specimens had all suffered a loss in stability, nevertheless the moisture content as determined by increase in weight was only moderately higher than in the unsealed specimens.

The tests were not comprehensive enough to be cited in detail to prove this point beyond question, but nevertheless results are in line with observations made in the field, i. e. that a relatively small amount of moisture entering a bituminous mixture in the vapor state may cause more damage than larger amounts present as capillary water.

Therefore, in original construction some thought must be given to the problem as to whether it is imperative to seal the surface against precipitation moisture or whether there is more water rising annually from below than falls as rain or snow. Many arid or semi-desert regions, particularly valley areas adjacent to high mountain ranges, have considerable ground water which apparently rises steadily, and the relative profusion of green flourishing desert plants on both sides of some of our oiled highways seems to prove definitely that they are drawing moisture from a reservoir created beneath an impervious pavement. One wonders if a judicious selection of roadside plants might offer the possibility of reducing excess moisture beneath pavement slabs. The ideal plant might be difficult to obtain, however. Desert plants have tremendous root systems and are very efficient in gathering moisture, but have little ability to dissipate the moisture from their foliage. Plants which are efficient in exhaling moisture from their foliage probably lack the far-reaching root system which would be necessary. It appears to be a problem for

the arboriculturist who might thus lend a hand in stabilizing subgrades of existing roads where reconstruction is not possible. It may be noted in passing that the amount of moisture evaporated into the atmosphere from a grove of trees is much greater than that from the same area of a lake or pond.

There are, however, other failures also attributable to subgrade conditions where the cause may be more obscure. Certain types of soil possess a considerable degree of resiliency when subjected to load. Such soils will have ample supporting power, and the road surface may show no sign of settlement or localized breaking up due to lack of support. Nevertheless, during the passage of heavy loads, the pavement and subgrade will be depressed and bending of the pavement slab will induce unusual stresses which have a marked tendency to develop surface waves or ripples in bituminous pavements. It appears that this is one subject which has thus far not been well investigated and further studies to measure the amplitude and frequency of slab movement should throw light on much trouble with water-bound gravel surfaces and bituminous mixed pavements.

(2) **Base Course Faults.** Base course faults may be treated rather briefly. In certain cases base courses may be subject to all of the troubles previously discussed in relation to subgrade. However, base course material is presumably better than the natural soil; therefore, the most frequent source of trouble is probably inadequate thickness. While there are a few examples of base courses which have been constructed heavier than necessary, there certainly are a far larger number of cases where the base course is inadequate.

One serious mistake that was once widely made in base course construction, but which is not so prevalent in recent years, is the use of comparatively clean rock without fines over subgrades which became plastic when wet. The mud readily works upward into the voids of the coarse rock and may completely destroy the usefulness of the base. This sort of thing has happened many times.

It is, of course, true that test requirements and specifications originally established from research can become outmoded, and as a result may constitute liabilities rather than assets. For example, 15 or 20 years ago water bound gravel construction was widely used and was intended to carry traffic without further treatment. It was of course essential in the horse and buggy days and in the earlier years of state highway construction that gravel roads be bound up tightly and hence materials needed cementing value. The need for high cementing value, however, is no longer pressing, and in many cases the requirement can be dispensed with. Nevertheless the test is still in use.

Some failures in gravel or crushed stone bases can be charged to the attempt to meet a cementing value requirement. There is a tendency to encourage the use of clay types of binder which may lubricate the stone when moisture contents are high.

One difficulty in placing graded crushed rock or gravel is due to the tendency of the material to segregate in the plant or during spreading and finishing. This difficulty is avoided by the slurry base, in which crusher run material uniformly graded, with a definitely controlled per-

centage of dust passing 200-mesh, is premixed at the plant with sufficient water to provide thorough distribution of the cementing dust and permit ready compaction when spread on the subgrade. This practice eliminates segregation and provides a dense, hard "clay-concrete" which has no pores for the entrance of subgrade mud, and is hard and smooth enough to carry traffic during construction.

(3) **Aggregates Best Adapted.** Since the beginning of bituminous construction, ideas concerning mineral aggregates have undergone a certain amount of change. It is no longer considered sufficient safeguard to specify that an aggregate shall be hard, sound, durable and well graded. While it is still true that durability is required, it is not so widely believed that durability and hardness are synonomous. Investigations of existing projects furnish evidence that in the dense graded type of bituminous mixture, hardness of the rock is of minor importance.

The principal contribution of research toward reducing the problems of maintenance has been recognition of the fact that mineral aggregates have widely varying surface characteristics which affect their capacity to retain an oil film in the presence of water, and which mechanically may affect the stability of the mixture.

In the early days of bituminous road construction, frequent failures resulted, many of which were regarded as mysterious, and furnished ample grounds for controversial discussions among engineers. It was customary to applaud the fortunate engineer who chanced to be in charge of work which turned out well, and to greatly criticize less fortunate individuals who were trying to construct oil roads with materials which, in the light of present knowledge, were definitely unsuited for the purpose.

The researcher has uncovered a great many facts about mineral aggregates which were virtually unknown ten years ago, and one result of his efforts has been to place the blame for certain failures where it belongs—not on the head of the hapless engineer or superintendent in charge, but on the type of mineral aggregate used. It is now widely recognized that many mineral aggregates are not suitable for bituminous construction due to their tendency to lose asphalt in the presence of water. Such aggregates should not be used without protective measures. And researchers are now busily engaged seeking better or more economical means for protecting such aggregates and soils. Some success has been achieved but a great deal of work remains to be done.

In the meantime, most highway departments are trying to avoid unsatisfactory materials by accepting or rejecting the aggregates on the basis of preferential wetting or stripping tests. This precaution alone has certainly resulted in tremendous savings in maintenance costs. Most states probably have one or two sections of road in which the aggregates used are definitely hydrophilic. A study of the maintenance costs on these sections would doubtless show that impressive sums of money have been saved by the avoidance of materials of this sort. As a matter of fact, it is not impossible that bituminous construction using lighter oils might have been abandoned completely if this phenomena had not been recognized.

There is some reason to believe that the use of oil in road construction received a serious setback many years ago due to failures of this sort.

For instance, the Petrolithic method described by Prevost Hubbard in 1908 in the U. S. Department of Agriculture Bulletin No. 34 refers to the use of heavy asphaltic oil mixed with loose earth and compacted with a sheeps foot roller. While some of the projects were very satisfactory, others failed, as may now be surmised due to the use of unsuitable soil types. Apparently the method fell into disrepute as a result of a few failures.

The amount of technical literature now available and being written describing numerous investigations, methods of test and treatment applying to the subject of hydrophilic and hydrophobic aggregates makes it unnecessary to go into further detail concerning this now well-known phenomenon.

There is no simple means yet available for classifying or describing types of mineral aggregates which are best suited to bituminous construction. Petrographical and mineralogical classifications are useful for identifying certain aggregates, but suitability for bituminous work is not entirely determined by the mineralogical classification. Reports from various parts of the world show that granitic rocks may be either very good or comparatively poor; the same is true of limestone and other types. It is true that careful study will often uncover the cause why individual samples of a certain aggregate may vary in quality, but the point is that no definite prediction can yet be made in the vast majority of cases without resorting to test methods of some sort. Only a few broad generalizations can be made. Limestone and basalt or trap rock are for the most part very satisfactory. And quartz, chert and rhyolite, for example, may well be viewed with suspicion but one can never be sure in individual cases without tests.

Requirements for hardness of mineral aggregates may be varied to suit the type of construction. There seems to be ample evidence that comparatively soft aggregates may be used without danger in the dense graded type of construction. The open graded and macadam types require greater hardness.

(4) **Bituminous Materials.** Before taking up this phase of the discussion, one feels inclined to pause and draw a long breath. It is difficult to cite any important research applied to bituminous materials which has thus far borne definite fruit in the way of improved specifications which will guarantee bituminous materials better suited for the purpose of highway construction. Test procedures and limits which are commonly specified for the control of asphalts are for the most part of long standing, and have been used since the beginnings of bituminous pavement construction thirty or forty years ago. There is almost universal agreement that existing standard tests throw little light on the suitability of oils and asphalts for road building purposes. This does not mean that the problem is being neglected; on the contrary, a great deal of work is being done and there is every reason to hope that within a comparatively brief time we will be in possession of a great deal more knowledge than we now possess concerning the essential character and composition of asphaltic binders for bituminous pavements.

The principle evidence for lack of quality in bituminous materials is the frequent deterioration due to cracking of pavements, raveling and

disintegration due to the asphalt becoming too hard and brittle, thus losing its capacity to resist shock and abrasion.

While the evidence is often contradictory, there is a high degree of correlation between over-heating and cracking of the asphalt in the refinery and failures due to hardening and brittleness on the road. Tests such as the Oliensis spot test have been offered to detect asphalts which have been damaged by over-heating. Unfortunately, some crudes which have not been heated will show the same characteristics in this test as do cracked asphalts. Recent modifications* of this test substituting normal heptane for Oliensis solvent appears to offer an improved method for detecting cracked products. The problem is being approached from other angles; for example, the work of Benson in Kansas, in which a thin translucent film of asphalt is subjected to weathering conditions and changes in the character of the film observed under high magnification.

Other investigations by means of accelerated weathering equipment and by exposure to natural sunlight are also being carried on, and studies of the bitumens are being made to find the cause for variations which exist.

While it is true that most cracked products are numbered among the unsatisfactory materials, nevertheless an occasional oil which has all evidences of having been cracked will give good performance. One fact which adds interest to the problem is that many bituminous pavements built 20 to 25 years ago are still showing good service with no cracking or failures, while many recent jobs are in considerable distress from this cause. The trouble is not confined to certain sections of the country but seems to be fairly wide-spread. Two explanations may be offered—one that more asphalt was used in earlier days of construction when pavements were designed for horse-drawn traffic, and this practice changed when the advent of the automobiles brought justifiable fear of the skid hazard. At the present time, while most asphalt technologists agree that the quantity of asphalt in a bituminous pavement should be as high as possible consistent with other considerations, there is a great deal of difference of opinion as to how much is too much.

The same criticism that was made of cementing value tests may be launched at an over-emphasis on stability tests for bituminous mixtures. In past years a large proportion of bituminous pavements became rough and wavy sooner or later because of unstable mixtures, usually due to an excess of asphalt for a particular mixture. Stability tests are useful as a check on this condition. However, attempts to obtain the highest possible stability may lead to mixtures that are too dry and brittle to withstand the effects of weather and traffic.

The second explanation which may be offered can only be in the form of a suggested reason, as the theory is by no means proved. It seems probable that many asphalts produced in recent years have been subjected to refinery methods which for the purpose of producing lubricating oil and gasoline are more efficient than was the case 25 or 30 years ago. The removal or destruction of certain fractions which may act as protec-

* Proposed method for the modified quantitative Oliensis test using heptane, developed by the Standard Oil Company Laboratory during the California Cooperative Investigation.

tive colloids would produce a more unstable product. The asphaltenes would flocculate more readily producing a non-uniform heterogenous film as observed by Benson. There is, of course, little justification for speculation in an alleged technical paper. In any event, the problem confronting the asphalt technologist today is to devise specifications and methods of test which will definitely define the properties and composition of the ideal road building bitumen.

This particular chapter would seem to call for some learned discussion as to the relative merits of the numerous grades of slow, medium, and rapid curing products which are available to the bituminous engineer today. Practically everyone connected with bituminous highway construction has his preference or preferences, as does the writer. There appears to be no point, however, in strongly advocating the use of this or that type of material over another, inasmuch as progress in research has not yet advanced far enough to offer anything definite to the problem.

It might be remarked, however, that the best bituminous material to use is the cheapest one which will serve the purpose, and when this consideration is put to the test, it is surprising how few cases are definitely unsuitable for the various slow curing oils.

(5) **Special Problems.** One of the few special problems related to maintenance which have been investigated concerns the design of premixed materials for use in maintenance patching. In California a great deal of patching of bituminous surfaces is made by means of dense graded premixed material, stored in stockpiles. At intervals certain shipments of this material are found to be unsatisfactory, either failing to set up, to bond to the old surface, or to remain stable under traffic. It appears that a patching material must have a nice balance of properties, neither too fine nor too coarse, the oil can be neither too heavy nor too light, and the permissible aggregate gradation must evidently be kept within narrower limits than is necessary for new construction. For this reason, studies have been made in California by analyzing stockpiled mixtures classed as both satisfactory and unsatisfactory, for the purpose of discovering the essential requirements. While the work is not complete, and further proof is necessary before too positive statements can be made, the following grading tolerances are suggested which, with average types of mineral aggregates, should furnish satisfactory mixtures for maintenance patching. As stated above, these mixtures have not been used for a sufficient length of time to prove their quality beyond question. However, the gradations seem to avoid most of the types which have given trouble.

The following table gives the proposed gradations in terms of the U. S. Standard Sieve Series, A.S.T.M. designation E-11-26, A.A.O.S.H.O. Method T-27.

March 16, 1938

MATERIALS AND RESEARCH DEPARTMENT TENTATIVE GRADING TOLERANCES FOR DENSE GRADED BITUMINOUS MIXTURES

	¾-Inch	½-Inch	⅜-Inch	4-Mesh
1-Inch	100%			
¾-Inch	95%-100%	100%		
½-Inch		95%-100%	100%	
⅜-Inch	65%- 85%	80%- 95%	95%-100%	100%
4-mesh	50%- 65%	58%- 73%	65%- 85%	90%-100%
16-mesh	30%- 43%	33%- 45%	38%- 52%	46%- 67%
50-mesh	18%- 27%	18%- 27%	22%- 30%	25%- 36%
200-mesh	6%- 10%	6%- 12%	8%- 15%	10%- 17%
Oil Ratio	3.9 - 4.6	3.9 - 4.7	4.2 - 5.0	4.5 - 5.3

(For average types of aggregate)

The most satisfactory grades of liquid asphalt for premix patching material have been SC-4, ROMC-3 and MC-2. In California, preference has been in the order stated.

Progress in Research on the Improvement of Soil Properties by the Addition of Granular Materials, Tar, Asphalt and Portland Cement

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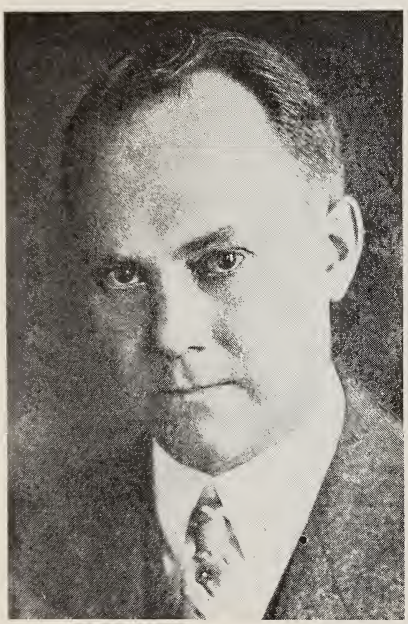

Just where investigation and experimenting leave off and research begins is difficult to say. In our work on soil altogether too much has been said for research when, as a matter of fact, all that we have been doing is snooping into the outer fringes of Nature. However, since most of our snoopers like to dignify their experiments with the term research we shall gather together such facts as have recently been presented under that name.

An interesting machine known as Brabender's Plastograph has been made suitable for testing soils by E. O. Rhodes of the Koppers Company. This machine measures continuously and records viscosity and plasticity. Highway soils at present are tested by time-consuming methods which are often of questionable accuracy and significance. Several empirical tests are used to determine the proportions of admixtures required for soil stabilization which do not always indicate correctly the amounts required for best service results. Brabender's plastograph measures consistency by means of a dynamometer. It simplifies soil testing, gives better reproducibility of results and furnishes needed information as to the effects of admixtures. Other possible uses for the machine are suggested.

*Donald M. Burmister, Columbia University, says that the distribution of particle sizes and particle shapes have a very important influence on the density of materials, and therefore special consideration is given to the grading of soils in order to bring certain physical factors into a more unified and consistent pattern. Density is influenced not only by the degree of fineness of the material and the range of particle sizes, but also by the distribution of particle sizes, expressed in "type grading curves." These factors, together with the effect of particle shape, are

* The Grading-Density Relations of Granular Materials by Donald M. Burmister, A.S.T.M. proceedings, 1938.

expressed quantitatively in terms of grading-density relations, which provide a means of estimating the influence of grading on density. These relations are important because density is an important factor in the supporting capacity of the natural soil and in the stability of slopes of granular soils. In his paper presented at A.S.T.M., Atlantic City, June 1938, Burmister presents his findings in detail.

E. E. Bauer, University of Illinois, has begun a study on deflocculating agents used in the particle size determination of soils. The abstract from his paper† states that "most of our soils flocculate when placed in water —that is, particles gather themselves together in clusters. These clusters have an entirely different settling velocity than the individual particles would have and makes any determination of their sizes by a sedimentation process impossible, unless something can be done to prevent flocculation. A number of deflocculation agencies are employed, such as sodium silicate, potassium hydroxide, ammonium hydroxide, sodium carbonate, and sodium oxalate." Bauer has made a study of the problems involved and his paper presents his findings so far.

The U. S. Bureau of Public Roads exhibited at the American Road Builders Association meeting in Cleveland a series of 20 panels which are excellent. They show in sufficient detail nearly all that we know today concerning road stabilization. This series has been reproduced in booklet form, and should be in every road builder's possession. They have been produced in short form in the proceedings of the Highway Research Board, 1937.

Hogentogler and Allen presented a paper in 1938 before A.S.T.M. on the application of soil mechanics to stabilization. This, together with their individual papers on the subject, form a valuable aid to the novice, and a restraining rein to the so-called expert. For instance, Hogentogler says "the best of natural soils should be selected for use in stabilized soil bases and road surfaces. The plasticity tests, with the mechanical analyses, serve excellently as guides in the selection and the blending of road soil mixtures. The plasticity tests do not indicate the presence of semipermanent or permanent cements. Certain rock powders may have a plasticity index of 0 and still have high cementing value when chemically treated. The grading requirements likewise are not critical when materials have the more permanent cements . . . Plant mixing, under rigid control, seems preferable for the more complex blending of aggregates soil and chemical solutions or insoluble binders . . . This should be a profitable field of investigation for the mineral aggregates industry. Fine graded soils, with commercial binders as stabilizers should be mixed and compacted with carefully determined moisture contents to specificied densities. The Proctor plasticity needle is essential for control purposes in this type of stabilization."

Henry C. Porter, Texas State Highway Department, has contributed a paper to Highway Research Board on "Improvement of Subgrades," wherein he uses the word subgrade to indicate the prepared surface of the soil substructure or any part thereof. He says that many a pavement failure may be attributed to the placing of granular materials on clays in such

† A Study of Deflocculating Agents Used in the Particle Size Determination of Soils by E. E. Bauer, A.S.T.M. proceedings, 1938.

fashion that water is impounded. Further, that there is bulking of damp sand when disturbed or manipulated.

His paper, and that of G. A. Rahn on "Materials and Design of Stabilized Soil Mixtures" are published in the 1937 proceedings of Highway Research Board. Mr. Rahn is with the Pennsylvania State Highway Department. Using the soil mortar gradation chart he gives five designs of road mixtures which cover a wide range of possibilities. He states that the soil mortar gradation is the heart of the stabilized soil road, and in all cases should be of the right composition. "Coarse graded materials are added where practicable in all cases whether the best coarse material is available or not, the voids in the coarse material should be filled with properly graded material." Now this sets up some discussion among us because Hogentogler states in Bulletin 47, A.R.B.A., that "the whole theory of soil stabilization is based on two facts: First, that all soil mixtures commonly used for road surfaces and speedways are highly stable at some moisture content; and secondly, that the moisture in the stable roads is more adhesive than free water." Here we have two men who disagree apparently on the basic fundamentals of soil binder stabilization.

A. T. Goldbeck, National Crushed Stone Assoc., has run some interesting experiments on the stabilization of soil with stone screenings. In his paper presented before the Association on Jan. 24, 1938, Goldbeck states that "it is our conception that a stabilized screenings base for highways should possess the following characteristics:

1. It should transmit superimposed wheel loads to the sub-base without appreciable movement and with very little deformation.

2. It should be able to perform these vital functions under all conditions of weather and moisture.

3. In the event of partial failure under favorable subgrade conditions, it would seem desirable for the best results that stabilized screenings should have properties which would permit it to heal or reknit itself together again. This point may be controversial.

4. It should remain stable, even after evaporation of capillary moisture has been prevented by the application of a bituminous wearing course.

"Logic would lead one to the conclusion that the maximum stability under wet conditions would be obtained, first, by using particles so graded in size that they will closely interlock one with the other and will provide a mixture having the greatest possible density and the smallest amount of porosity or percentage of voids; second, by the provision of just enough fine cementing material of some nature to aid in sticking the coarser materials together; third, by either integrally waterproofing the layer so as to exclude excessive quantities of water or by so controlling the amount of water that excessive film thickness of water will not be built up." Here Goldbeck seems to be stabilizing the papers of both Hogentogler and Rahn, thus making them agree with each other. Preliminary studies were made of the grading of the materials which would produce the most stable mixtures, and also of the amounts of bituminous materials to use. Then the mixtures were laid on a circular track

in the laboratory and subjected to the passing over them of a loaded wheel. Goldbeck goes on to say that "high stability is easily obtained when the mixtures are in dry condition, but that when in a wet condition there is a wide range in stability. Apparently gradation is the most important single factor in bringing about a stable condition under bad moisture conditions. The use of waterproofing materials such as tar, cut-back asphalts, and emulsions is not effective unless the mixtures are dense and have a certain amount of fine dust present. . . . . All of the mixtures in their dry condition still had a certain amount of moisture in them and yet they were stable. When this moisture content was increased they became unstable in many cases. There is a certain optimum moisture content beyond which instability occurs. . . . It seems that the best stability occurs when the aggregate is graded to give high density and when there is sufficient bituminous material present to make for low absorption and to furnish some resiliency to the mix."

Goldbeck's experiments are most interesting and throw much needed light on some phases of stabilization. They tie in many of the isolated instances of other investigators.

The Report of Committee D-18 of A. S. T. M. should be consulted. This committee is large, and the men constituting it are whole heartedly interested in the work of soil stabilization.

Bulletin No. 47 of the American Road Builders Association gives some good information and advice. In it we have the following articles: Role of Soil Binders and Aggregates in Soil Stabilization," by C. A. Hogentogler and J. A. Kelley; "Road Stabilization," by L. L. Allen; and "Stabilization with Aggregates, Binder Soil, and Calcium Chloride," by Fred Burgraf. While there is nothing in these papers that may be termed exceptionally novel, still, they present the subjects in readable form, and there is much to be learned from them.

BY THE ADDITION OF TAR

The important role that tar has thus far played in the stabilization of soil is too well known to require any further comment here. However, during the past year some interesting developments have taken place. The Barrett Company and The Koppers Company have done extensive experiments with tars to determine the kind and percentage to use with various soils. C. H. Olmstead, The Barrett Co., presented some of these findings in Contractors & Engineers Monthly for May, 1938. He showed how the laboratory work tied in with the practical work of stabilizing roads in Shelby County, Tennessee. A report was made by the laboratory recommending the grade of tar best suited for the soil, quantity required, optimum moisture of soil-sand mix, and advising a dampening before the application of the tar. The results were gratifying, and go to show how much can be accomplished by laboratory control, if the men in the laboratory know their business.

It would be well to present here the views of B. A. Anderton, Research Chemist, The Barrett Co., on soil stabilization methods and tests. "It should be emphasized that the methods, particularly those involved in the evaluation of treated soils, are subject to modification and to replacement by improved methods as they are developed. Thus far there

has been no consistent effort to standardize laboratory methods for evaluating stabilization nor for determining relative efficiency of varying treatment, especially as applied to bituminous materials. In fact, there is no general agreement as yet on the relative importance of many factors in soil stabilization and on the interpretation of test data, with the result that various laboratories are using different methods for evaluating stabilization and determining the best treatment for a given soil. In view of the present stage of development, it must be freely admitted that all methods for studying soil stabilization with bituminous materials, including those at present used in our own laboratory, are open to criticism. It is therefore not advisable when submitting a description of Barrett methods to imply that they are better than methods used by other laboratories, since those methods also are of value in investigating soil stabilization, and probably yield data of equal value. We consider that a recommendation on stabilization with tar will include:

1. The necessity of desirability of modifying the grading of existing soil;
2. The grade or grades of Tarvia for use in the mixture or mixtures.
3. The preferred amount of Tarvia;
4. The use of water in mixing;
5. The moisture content desirable for most efficient compaction.

In this discussion we have considered essentially only those soils which contain large percentages of fine material, excluding those soils which may be regarded as sand and susceptible to utilization in sand-tar mixtures.''

Mr. E. O. Rhodes, Koppers Company, has done some excellent investigating and his discovery of the suitability of the Brabender Plastograph for testing stabilization changes makes it seem, to quote one of the onlookers at a demonstration, ''as if lots of these college professors will be out of a job teaching their students how to measure viscosity and plasticity.''

ADDITION OF ASPHALT

E. F. Kelley, Bureau of Public Roads, in a recent paper* notes that ''given granular materials of suitable grading and soil binder of suitable quality it is possible to construct satisfactory stabilized base courses of the mechanically stabilized type. But in many localities the materials necessary for this are either not available or can be obtained only at an extremely high cost. From the economic standpoint it is here that bituminous materials enter the field of stabilization. . . It has been demonstrated that (they) are effective for the purpose.''

It is here, then, that the asphalt producers have apparently had their big opportunity, in which, in some cases, they have used good judgment, and in others they have been content to sell whatever they had on hand. Thus we find all sorts of materials being prescribed by different concerns for the same purpose. The more responsible companies have gone about the selling of their wares in a properly conservative spirit, and have not tried to push goods that are unfit for stabilization. Others,

probably thru ignorance, have endeavored to palm off any or all materials they make. This is most reprehensible.

H. C. Weathers,† Florida State Road Department, says "the types of asphaltic materials used in this construction have been cut-backs, mixtures of heavy asphaltic oil and light distillates, emulsified asphalt, and flux oil with hard powdered asphalt. The asphaltic material used most generally and with the best success has been cut-back asphalt."

Mr. Weathers' statements of construction methods are interesting, and they indicate what Florida has been accomplishing in its stabilization work. He describes the blade mix which is done with powdered asphalt. "The flux is first added to the road material by a distributor, and is cut in with harrows; then the hard powdered asphalt is applied in proper quantities, and mixed with harrows and graders until a uniform mix is obtained."

A. C. Tilley,‡ Nebraska Department of Roads and Irrigation, says that "in the design of mixtures and in field control, the findings or conclusions that have been arrived at thru experimental projects and laboratory research, are carefully heeded. Some of the most important of these are as follows:

1. In order to stabilize properly the sandy soils with asphalt, it is necessary to use the same care in the selection of fillers or aggregate admixtures and in the proportioning of materials as is used in the design of dense-graded bituminous mat mixtures.

2. When bituminous sand surfaces are constructed directly upon dune-sand subgrades, it is necessary to provide 5 to 6 inches of stabilized sand having a stability value of from 300 to 400 pounds at 77° F. in order to distribute wheel loads over a sufficient area of the sand subgrade to prevent its displacement.

3. When this type of surface is constructed on sandy loam with 20 to 30 percent finer than the 200 mesh sieve, a thickness of 3 to 4 inches is required having a stability value of from 600 to 700 pounds at 77° F. in order to carry properly the loads imposed on it.

4. It is more economical in most cases to rectify the gradation of the soil rather than attempt to use special asphaltic materials or to use a high percentage of oil in the mixtures.

5. The percentage and type of filler material in the mixture is the chief influence on the stability of the mixture and is the principal factor affecting the quantity of oil required.

6. The use of MC oils permits a greater variation in the mixture without endangering satisfactory results than does the use of SC oils because of the higher stability and cementitiousness provided by the MC materials. These qualities permit the use of inferior gradations and higher oil percentages when necessary than is possible with SC materials."

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*Basic Principles and Economics of Subgrade Stabilization with Bituminous Materials, E. F. Kelley, Proceedings of 11th National Asphalt Conference.

†Stabilizing Sandy Soils with Asphaltic Materials, H. C. Weathers, ditto.

‡Stabilizing Sandy Soils in Nebraska, A. C. Tilley, ditto.

Data of different nature is given by F. V. Reagel, ‡Engineer of Materials, Missouri State Highway Department. He notes that in his state SC and MC type oils and cut-backs, crude oils, tars, and emulsions are used as treating agents. "The essential difference between our general procedure and that used elsewhere is that in our case the soil phase is the major phase instead of being present in a limited amount as a binder. Our mixes contain as high as 65 percent passing the 40 mesh sieve, and in such mixes the aggregate present is rather an adulterant than a friction producing material, or at least is much more free to move if lubrication is produced. Accordingly, our optimum quantity of treating agent is smaller (about 7 to 8 percent by weight of the soil fraction) and the film thickness is so reduced as to resemble a stain rather than a binding film. Consequently, plasticity is avoided and a well graded aggregate is not required to overcome the plasticity introduced by excess quantities of bitumen."

L. H. Taylor, *Maintenance Engineer, Oakland, California, gives some interesting details concerning treatment of quarry waste with emulsified asphalt in which he gives the requirements for emulsified asphalt as follows: "Emulsified asphalt to serve properly as a stabilizing agent must have certain definite characteristics. It must (1) a mixing type, (2) miscible with water in all proportions, (3) capable of mixing with clay or the finest rock dust without balling up, (4) must be able to lose its water vehicle rapidly by dehydration after complete mixing with the aggregate has been accomplished, and (5) after dehydration must leave the remaining asphalt uniformly dispersed as a film in such a way that the material will retain all its cementitious binder characteristics and, at the same time, remain relatively immune to loss of cohesion from water absorption.

"The quantity of emulsified asphalt required to effect a proper stabilization is based on that material in the aggregate passing a 200 mesh sieve. Materials proposed for use in stabilization should also be checked for the approximate amount of detrimental water-soluble salts present."

These papers are interesting and yet they do not tell the whole story. For we have no examples of the failures that have resulted from attempts at stabilization. All too frequently our reports tell the happy ending and disregard the other side. The writer here would like to include some of the unhappy results which he has observed in another connection both in the field and in the highway engineering laboratory of the University of Illinois. In 1932 a number of samples were made in which various bituminous materials were used with several widely differing aggregates in order to observe the binding value of the bitumens. Altho this experiment has been reported it would be well to say a few words about it here. Five year tests now have been run.

It was observed at first that there was little difference among the samples stabilized with any of the asphaltic materials. There was, however, considerable difference between these and the ones to which tar had been added. The tar specimens hardened rapidly, and in six months

‡ Bituminous Stabilization Methods for Use in Missouri, F. V. Reagel, Proceedings of 11th National Asphalt Conference.

*Treatment of Quarry Waste with Emulsified Asphalt, L. H. Taylor, ditto.

were, for the most part, very hard. Those made with cut-back tar became hard and brittle. Those made with a small percentage of light coal tar did not become brittle for three years or more, and those made with 7 percent or more of light coal tar are not brittle now, after six years.

Among the specimens treated with asphaltic materials those made with cut-backs are best today. It was noticed that specimens made with emulsions hardened on top so that the upper layer powdered away, and that they were soft and soupy underneath. Evidently where the air did not come in direct contact with them the emulsions never "broke." This was also observed by the writer and others on a road job in Ohio where several different emulsions were used. It has also been observed elsewhere and by other investigators. This same thing has been seen to exist with some of the cut-backs and road oils as well. In other words, there is a hardening of the outer or upper skin, if we may call it such, which prevents a hardening of the lower layers, much as paint skins harden on the exposed surface of an uncovered can of paint. Therefore, the road material is not stabilized underneath in many cases, but retains a pasty to soupy consistency for a long, long time. This condition needs to be corrected, it needs investigating, it needs **research.**

Another matter that the writer has brought up with several of the producers of stabilization materials is the value of the shear test. It has been apparent that the original investigators have set great store on shear as the answer to the problem of stabilization. Many pages are filled with formulae for figuring shear. Yet, the interesting fact is that all too frequently the mathematically derived answer does not agree with observed results as to the behavior of the soil in field tests. What is wrong? The engineer has been treating soil as tho it were a structural member of a bridge, and—it isn't.

The engineer made a similar error years ago concerning asphalt, and condemned its use for the prevention of erosion along the Mississippi.

BY ADDITION OF PORTLAND CEMENT

Much light has been thrown on the cement stabilized road by the recent publication of the book entitled "An Experimental Soil-Cement Road in Illinois," published by the Division of Highways of the State of Illinois, April 25, 1938. The subject is gone into thoroly, tables are given, and the trials and tribulations of the experimentors are mentioned. Much has been learned from this road, constructed in 1936, and those who are interested in this type of stabilization should procure a copy of the report. It is apparent from the laboratory tests that those samples containing 10 per cent of cement stood up far better than those with lesser amounts.

On page 13 the following conclusions are given:

(a) Preliminary samples on which the job control data are to be based should never be taken until the grading operations have been completed.

(b) Extreme care should be exercised in taking samples on which the job control data are to be based. The location at which the samples

are taken should be carefully selected and a sufficient number of samples secured to represent satisfactorily the soil types and variations within these types.

(c) The equipment for preparing the soil, mixing the cement, distributing and incorporating the water, and compacting the mixture, should be such that the actual time of processing will be reduced to the minimum.

(d) Comprehensive field tests should be conducted during the progress of the job.

It will be interesting to note the tests of W. F. Kellermann and D. G. Runner in their paper presented at the 1938 A. S. T. M. meeting, on "The Effect of Using a Blend of Portland and Natural Cement on Physical Properties of Mortar and Concrete." It is possible that such a blend as they suggest, used for stabilization purposes, may improve the resulting mix to an appreciable extent since the resistance to freezing and thawing is materially increased.

The author has collected the most recent information published in the United States. If one talks to some of the representatives of the many producers of stabilization materials he will find that there is a reluctance to make any really definite statement concerning positive stabilization of earth roads. There is considerable groping about in the dark, as it were, and no one in a responsible position wishes to go on record as to being able to produce certain and sure results of a worthwhile nature. This is as it should be, and it will not be long before we shall be able to certainly predict what the outcome of any one treatment may be.

Research Developments Related to Service Behavior of Asphaltic Materials

By

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Within the past several years, much has been said and written regarding the need for more extensive research in the field of asphaltic materials used for highway purposes. Engineers and chemists concerned with these materials have long complained that the conventional laboratory tests employed for specification purposes are inadequate to insure the desired quality and serviceability and there has been a consequent tendency on their part to create entirely new tests or to modify the conditions under which long established laboratory tests have heretofore been conducted. Mr. E. F. Kelley, of the U. S. Bureau of Public Roads, in discussing (1) the resulting chaotic conditions some time ago indicated that we were no further advanced than we were forty years ago in being able to anticipate the service characteristics of an asphalt by means of rational laboratory methods. The Highway Research Board, referring to the subject in one of its publications, (2) emphasizes the fundamental need for more research but suggests first an examination of the record as a desirable preliminary to future study. The producers of asphalt are, at least, on record (3) as to the present confusion of test requirements and some of them have been frank enough to show the futility of attempts to insure quality under certain present day specification practices (4) (5).

As these conditions have been brought before our various technical societies, discussions have usually led to the establishment of committees to give them detailed study in the hope of finding some solution of the problem. Thus, in recent years, there have been set up the following groups which are concerned either directly or indirectly with this question of quality or service behaviour of asphaltic road and paving materials:

Specification Committee on "Bituminous Pavements" of the American Public Works Association.

Sub-Committee C-3 on "Road Oils" of Committee D-4 American Society for Testing Materials.

Project Committee 3-A on "Asphalt Cements" of the Association of Asphalt Paving Technologists.

Project Committee 3-B on "Liquid Asphalts" of the Association of Asphalt Paving Technologists.

Sub-Committee on "Improvements in the Form of Asphalt Specifications" of the Technical Advisory Committee of the Asphalt Institute.

Committee on "Characteristics of Asphalt" of the Highway Research Board.

These activities, many of them overlapping and duplicating in both scope and personnel, have already entailed much time, patience and expense on the part of the various interests represented for committee work proceeds slowly at best and it is even difficult to get such committees together. The net result to date, however, is that, while much progress has been made in the standardization of test methods, coordinated research remains neglected and the purchase of asphaltic road materials is still surrounded with confusion and uncertainty. To what, then, are these unsatisfactory conditions due? I am sure that a candid examination of the subject will show that they are due primarily to a combination of conditions which have been created by the changing practices of both producing and consuming interests and for which both are responsible.

In tracing the history of asphalt paving, one does not have to go very far back to find that the requirements, not only for asphalt, but for the manufacture and laying of the paving mixtures themselves emanated from the producers of asphalt. Indeed, for many years, the producers of asphalt also constructed the pavements, usually under performance or service guarantees to the cities and municipalities, a system which was still in effect as late as the year 1912. It is not here intimated that these early asphalt specifications provided, in themselves, any assurance as to quality or serviceability but rather that this combined responsibility of supplier and contractor tended to promote a more intimate knowledge of the individual asphalts then available, and their appropriate combinations with mineral aggregate, than prevails under present day practices.

Within the space of a relatively few years, however, as highway construction advanced to the status of a major industry the drafting of specifications passed entirely from the material interests and a few independent consulting laboratories into the hands of state and municipal officials. The asphalt supplier's function became that of simply meeting the asphalt specification requirements while the contractor, not any more bound by guarantee, has been placed in a position where he must, naturally, obtain the lowest price asphalt which will pass the necessary inspection. Is there any wonder that fundamental research, both costly and uncertain in its financial returns, should have fallen into a sort of "no man's land" in so far as any definite and coordinated program was concerned?

With the increasing use of asphalt for highway purposes during the transition period referred to, there were, also, rapid and important advances in the design and operation of refining equipment with the result that batch stills have been practically displaced by continuously operating vacuum equipment. Some of the more efficient installations of the latter type are capable of recovering asphalt from almost any crude petroleum which carries this constituent in appreciable quantities and most of these products have apparently found sale under the ordinary open type of specifications prevailing. That these asphalts, prepared from an increasing number of new and unproven sources of raw materials, could

possess vastly different inherent qualities, not shown by such specifications, and consequently give different results in service, did not become a matter of serious concern until comparatively recent times. The highly competitive system, with its divided responsibility, had created the impression that all asphalts complying with a given set of requirements could be used interchangeably, grade for grade, and still give the same results in service. One needs only to study the work of Mr. H. W. Skidmore[6] to observe the great variance in behavior of some twenty paving asphalts of 40-50 penetration under low temperature conditions. It is reasonable to suppose that, in the field of low cost road materials, differences in properties may be even more pronounced.

Until very recently, attempts to write more adequate specifications to support the open competitive system consisted chiefly in adding so called identification tests to the conventional physical and chemical requirements and, under this practice, such laboratory determinations as paraffine scale, fixed carbon, sulphur, naphtha insoluble, fluidity factors, and more recently, low temperature ductilities came rapidly into use. Specification limits of such test requirements are usually adjusted on the basis of some material or materials which have been used successfully and with the idea that the low bidder under such a specification will supply such products. The fallacies of these practices are many. Identification tests offer, at best, but a thin disguise for the closed specification actually intended and, in effect, are just as much against public policy as specifying a preferred material by name direct. On the other hand, producers of out-lawed materials have been encouraged to resort to various mixtures, blends and concoctions which often succeed in meeting these so called "trick test" requirements. Some producer representatives have, indeed, already told with amazing frankness of their success in these directions. Asphalt manufacture has, therefore, been encouraged to become, to a considerable extent, a matter of devising imitations rather than developing normal products from definite raw material and marketing them on their merits. I seriously doubt if the public has been better served under this system than it would have been under a more frank method of dealing with the subject.

The first important step toward a more rational method of procedure in the examination of asphaltic materials for highway purposes was doubtless taken in connection with a survey of state highway specifications for liquid asphaltic products which was undertaken in 1930 jointly by the U. S. Bureau of Public Roads and The Asphalt Institute. This survey was a preliminary to the extensive program of simplification, which was subsequently carried on by the states and the industry under the sponsorship of the Bureau, to eliminate the confusion of results being obtained in the low cost types of road construction. Confronted with over seven hundred products in current use, under about two hundred specifications, the executive committee in charge of this preliminary survey early realized the imperative need of a more simple yard stick to replace the one hundred and nineteen different tests employed for evaluating these materials. As a consequence, the now generally recognized distillation test was promulgated.

As is well known to most of you, this test provided a laboratory method, more or less arbitrary it is true, for separating volatile constit-

uents from base materials yet rational to the extent that the separation this accomplished revealed substantially the constituents employed by the producer. Instead of applying a number of meaningless tests to unknown blends or mixtures, therefore, their essential components would be made available for separate study as to their properties. This revolution in testing enabled us to classify all of the liquid asphaltic road materials into three groups or types, rapid, medium, and slow curing. It thus served to clear the air of much of the mystery and confusion surrounding the use of these materials and it provided a basis for specifications which has since come into extensive use. Unfortunately the scheme, although rational so far as it went, offered nothing new when it came to determining the quality or serviceability of the distillation residues and this has proved especially disconcerting in the case of the soft and liquid base materials. It is believed, however, that the usefulness of this important departure from previous practices will become more and more enhanced as our research expands.

Another step in the transition from purely identification tests to tests indicative of quality occurred in the year 1933 when Mr. Oliensis presented to the industry (7) the results of some seven years research in what he has termed the relative heterogeneity of asphalts. Differentiation between "homogeneous" and "heterogeneous" asphalts is accomplished by means of the so-called "spot test" now too well known to require detailed description here. Although the author of the test has made no claim as to its being a yard stick of either quality or serviceability, it afforded, for the first time, a direct insight into the heat treatment accorded the asphalt during refinement. Thus, materials subjected to excessive temperatures react "heterogeneous" by showing a black spot or nucleus when the naphtha dispersion is dropped on paper or glass while materials produced under more moderate temperature conditions are recorded "homogeneous" by showing a uniform brown stain or clear varnish-like film on paper and glass respectively.

While the test is sufficiently sensitive to identify asphalts that have been only slightly and, perhaps, unintentionally overheated, the heterogeneous group includes all of those products prepared from the residuals of cracking operations in the production of gasolines. Coming out at a time when these cracked asphalts were being so extensively and, in many cases, surreptitiously introduced into the highway industry, there is little wonder that the Oliensis test filled a long felt need and that it was pressed into specifications (8) with practically no preliminary or probationary period. As in the case of all test methods, some troublesome border line cases have arisen and, until the procedure is more completely studied and standardized, careful discretion and judgments should be exercised in the interpretation of such instances. In a more recent contribution (9) the author of the spot test has provided for determining the degree of heterogeneity by means of varying mixtures of naphtha and xylene instead of the naphtha alone as used in the basic method. One is now able to determine, for instance, the extent to which an asphalt has been "cracked" and herein lies the possibility of explaining why these by-product residuals have performed with such varying degrees of success in highway construction.

The direct exposure of asphalts to the weather early attracted the attention of investigators in the field of asphalt paving and we have the work of Hubbard and Reeve (10) and Reeve and Lewis (11) as examples of such studies. It remained for the asphalt roofing industry, however, to develop an accelerated laboratory test for this purpose. As early as about fifteen years ago the manufacturers of asphalt shingles and roofing recognized the fact that all asphalts complying with a single specification did not behave the same when subjected to weathering in the form of finished roofings. Under the sponsorship of their technical committee a research program was instituted and the result has been the adoption of a method (12) which has become standard in the industry. Under this procedure films of asphalt, oxidized to conventional requirements as to melting point and penetration, are exposed to alternating periods of ultra violet light, water spraying and freezing temperature These films, of definite thickness, are spread on aluminum panels which are carried on the inner surface of a vertical cylinder around a standard arc lamp which is the source of the violet light rays. Break down is determined by the ability of an electric current to pass through the disintegrated film to the panel backing. Many of the leading roofing manufacturers now require that their asphalts shall withstand a minimum number of cycles in the accelerated weatherometer in addition to meeting the usual chemical and physical requirements of former practices. A further development of the test (13) has been the examination of the residues recovered from the water sprays, this showing the production of appreciable water soluble materials as the result of light action. The usefulness of this test to the roofing industry has encouraged the belief that possibly some test of this kind might prove desirable in connection with paving materials.

In 1932, Lewis and Hillman (14) subjected a series of twenty-two samples of liquid asphaltic road materials, of the slow curing type, to the action of air, light and solar heat. Thin films of these products were exposed in shallow trays under quartz glass; washed and dried air was drawn over their surfaces. No attempt was made to apply freezing temperatures and, on account of the fluid character of these materials, the action of moisture obviously had to be omitted. Changes in physical and chemical characteristics were determined periodically and the altered materials were also tested in admixture with mineral aggregate. Many important conclusions were drawn from this study but the most outstanding development was that it has discredited so many of the laboratory tests used in evaluating these materials. Indeed one commentator stated that the work seemed to warrant the scrapping of all of our previous conceptions as to this class of road materials.

Since the work of Lewis and Hillman, the accelerated weatherometer of the roofing industry has been modified for use with paving and road building asphalts (15). In this form of the apparatus arrangements have been made for carrying the materials under test in shallow trays on a horizontal revolving table instead of on panels affixed to the inner wall of a rotating cylinder. This change, of course, was made necessary by the fluid nature of practically all paving and road asphalts as compared to the stiff blown asphalt of the roofing industry. In the modified form, also, two standard arc lamps are provided instead

of the single one in the case of the roofing instrument. One of these lamps is located over the center of the revolving table while the other is placed in a corner of the cabinet just outside of the rim of the revolving table. The trays containing the specimens are placed around the periphery of the table and thus pass between the two lamps.

Apparently little if any work has been done on paving asphalts with this form of weatherometer but some preliminary studies conducted by the writer were rather discouraging. A set of samples comprising RC, MC and SC grades, as well as soft binders and asphaltic cements from various sources, failed to show any characteristic break down after several months of exposure due probably to the films being too thick and consequently absorbing any decomposition products and deferring indefinitely the end point. Another disappointment was the inability to introduce the waters sprays due, as already stated, to the fluid character of the materials under examination. It will be interesting to hear if others have attempted the use of this type of weatherometer in their studies of the liquid asphaltic road materials but it is the writer's view that, on account of the nature of these materials and their complexity as to grades and consistencies, its adaption will be found rather difficult.

In December of last year, Mr. J. R. Benson, in his paper entitled "Microscopic Reactions in Translucent Asphaltic Films" (16) presented an extremely valuable contribution in the way of an accelerated weathering test for asphaltic road materials. As its title indicates, the author's studies are based upon the exposure of extremely thin films of the asphaltic materials to various kinds of light, different temperatures, and combinations of these conditions, then examining these films under the miscroscope. By this procedure, we are now able to explore, in a relatively short period of time, such changes as coagulations, flocculation, pitting, hardening, wax crystalization and other alterations which doubtless take place under conditions of actual service. Indeed, the correlation of some of these observations with actual service results has already been indicated. It will probably be found difficult, if not impossible, to establish hard and fast standards under this method of evaluation and miscroscope readings are, of course, dependent upon empirical judgment and interpretation. In the writer's opinion, however, it offers a most promising means of anticipating the probable service behavior of asphaltic road materials.

To conclude this brief discussion, it may be safely said that progress has been made in asphalt research in spite of the many obstacles and, furthermore, recent efforts toward the anticipation of service behavior of asphaltic road materials are apparently proceeding along more rational lines than formerly. The much desired objectives are, however, far ahead and cannot be reached quickly and without cost. To encourage research with the idea of simply developing some miracle quality or service test will not only fail to solve the problems but will probably lead to disappointment. Competitive materials, under any specification, will probably require variations in technique for proper handling and this feature seems to have been greatly neglected. Much work therefore remains to be done on the individual asphaltic materials.

Research accomplishments to date have been largely the result of individual effort and, consequently, more or less disconnected. The writer

called attention to this situation as far back as 1929 (17) suggesting the advisability of pausing to take stock of our accumulated knowledge so that we might leave to future road builders some basic principles rather than a growing confusion of individual experiences. Essentially the same thought has been expressed in resolutions of the Montana Bituminous Conferences and the Eleventh Asphalt Paving Conference. It was also reflected in the proposal for a preliminary fact finding survey by the Highway Research Board mentioned in the opening statement of this paper. Action awaits only the provision of adequate financing and this should be the responsibility of all groups concerned with the manufacture and use of asphaltic road materials.

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Research as Related to Quantities and Types of Bitumen Used in Bituminous Surfaces

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The bituminous surface proper can be considered as composed of a prime, mat, (or wearing course), and a seal. The choice of quantities and types of bituminous material used in each portion of the bituminous surface is highly controversial, but through the conflicting arguments certain standard practices are discernable. Most of these established bituminous highway design and construction practices such as the choice of the type and quantity of bituminous material are covered by numerous highway manuals, specifications, and published reports. These practices are a combination of rationalization and deductions, often previous to research. The justifications or the error of these practices in the light of recent studies are attempted.

PRIME

It is recognized by the highway engineer that the functions of the prime are:

1. To develop a film or layer with proper physical and chemical properties to make it impervious to moisture, thus protecting the mat above it.
2. To form a continuous bond between subgrade and mat.
3. To increase the effective thickness of the mat.
4. To consolidate the loose subgrade when required.

It is further recognized that the prime must be below a certain consistency depending upon the type of subgrade to obtain the proper penetration. The lower limit of this consistency is limited because too low a viscosity prime will penetrate too far, leaving a lean permeable prime.

Differences in penetration and action in use have been noted both because of differences as to type—that is, SC, MC, or RC—as well as the character of the bituminous base. That RC materials do not penetrate well, and leave a layer of free asphalt is recognized and is due to the solvent's being too light and either it evaporates or is preferentially absorbed, both mechanisms increasing the consistency too much for any further penetration. This prime consequently is of the membrane type, readily injured and offers a distinct plane of failure. However if the base is extremely absorbent and other materials continue to penetrate and consequently get lost the RC is recommended. Scoria material, discussed by Conkling (5) is a graphic example where only RC dried fast enough to form a hard residue in time to give a good impermeable membrane.

It is well known however that MC and SC materials can be best used as prime, and that all variables considered equally, MC is found most acceptable. (7) (20). Because of the measurable and definite curing properties of the MC it can be expected to set up due to both evaporation and absorption of the distillate, leaving the asphalt deposited as suitable prime in the subgrade surface. This definite setting up of the MC of course increases the cementitiousness of the prime, more thoroughly consolidating the subgrade, giving a better bond between subgrade and mat.

Quantities of prime materials vary depending upon the conditions from .1 to .5 gallons per square yard.

Available research data justifies the above choice made by the highway engineers and materials departments, both as to type and quantity. Studies of stabilized soil aggregate mixtures by Stoddard (34) demonstrate the optimum priming material necessary to maintain a suitable moisture content in the soil. Although the functions of the prime discussed are the exact reverse of a prime under a bituminous wearing course—that is, to keep moisture in the subgrade—the mechanics of this prime are the same and consequently Stoddard's information is of interest with respect to primes under bituminous surfaces. He has shown the merits of the MC cutbacks as compared to numerous other bituminous materials. He has concluded that on the particular subgrade investigated in his laboratory that .4 gallons MC-1 grade was the most suitable prime. Briefly summarizing some of his data we find that .4 gallons per square yard MC-1 permitted transfer of .243% loss relative to the loss of the untreated surface, as compared to 1.232 for MC-2, 1.251 for SC-1, 1.047 for SC-2, .303 for RC-1, .948 for RC-0, 0.512 for TC-2.

The low transfer of moisture with the RC material can be probably attributed to the formation of a thin continuous film of asphalt on the surface of the subgrade rather than a prime which is part of the subgrade. Naturally such a prime would not be satisfactory on other than very absorptive bases, because it is easily injured and would not offer a suitable bond of the mat to the subgrade.

Stoddard also shows that for application of 0.4 gallons per square yard that the MC-1 penetrates .557 inches as compared to .516 for SC-1 and .419 for SC-2, all for the given laboratory subgrade conditions. It is of interest to note that the MC-1 depth of prime is only 12% better than the SC-1, but its effectiveness as a prime against moisture is far better; i. e., ratio of .243 to 1.251.

Although Stoddard's work has been limited to evaporation from primed surfaces, the mechanism of the transfer of vapors through membranes indicates that all other variations remaining the same, the same primes will show relatively the same resistance to the passage of water itself.

The work reported by Kelley (13), Stanton and Hveem (32), Allen and Gibson (1), Benson (2), etc., on a superior adhesion of cutbacks and asphalts as compared to SC further verifies the highway engineer's choice of MC primes. It would seem logical that materials most resistant to displacement by water would be the best material to place in the prime. Further the above and other sources (35), indicate the higher the con-

sistency of the material the more resistance to displacement. Consequently MC materials which eventually develop a penetration asphalt base should be superior. Unpublished work by other research laboratories shows that the actual transfer of moisture across a membrane for a given asphalt is materially lower the higher the consistency. This would infer the superiority of the cutbacks with their development of a penetration asphalt residue.

Further deduction, partially substantiated by research, indicates that high gravity high asphaltene positive spot materials are better primes than the normal typical negative spot material. Available reports (28) (44), on the bituminous treatment of soils indicates superiority of these positive spot materials is very high and cannot be ignored to get the best results. The superior adhesion to mineral surfaces relative to water as shown by the various wash, adhesion and swell tests further substantiates this opinion. The lower permeability as well as the absorption of water and moisture (unpublished) of positive spot relative to negative spot materials further justifies serious consideration of MC materials of this type to obtain the optimum prime and waterproofing.

MAT OR WEARING COURSE

Proportion and type of bitumen in a mix are extremely important as they must cement the individual particles of the aggregate together to form a bond capable of resisting the direct forces of the traffic, contribute to beam action wherever possible, recement particles in case of rupture, resist water action, and weather properly.

Table 1 summarizes current practices as to choice of the type of asphaltic materials for various types of construction and aggregate (7). Examination shows a decided preference for MC and SC materials for densely graded road mixes, and MC or soft asphalt for densely graded plant mixes. MC or RC materials or various consistency asphalt cements are more generally used for the more open graded aggregates depending upon the conditions and type of construction. The above practices are discussed in detail relative to the various types and grades, types of constructions, aggregates, and quantities in the various procedings of the Montana National Bituminous Conference, Asphalt Paving Technologists, Highway Research Board, State and Federal Specifications, and Asphalt Institute manuals.

In general the MC has been accepted in preference to SC for densely graded aggregate. Further the trend is towards the use of higher viscosity liquid products and softer asphalt cements (7) (13). The proper quantities are still being applied on the basis of formulae, visual observation of the engineer, laboratory trial mixes using the Hubbard-Field Stability (9) Hveem stabilometer (32), or various modifications of shear tests (15) (30) (31), compression tests, etc. to establish the proper limits of stability. The trend is towards the use of the maximum amount of bituminous material that can be used without danger of shoving as it is felt that stability has been over-emphasized at the expense of other important properties such as workability and durability (9) (12). Better correlation of laboratory tests and service are being found. Adhesion is being more critically considered (6) (17) (43).

The preference for the MC at the expense of SC is generally substantiated by research. Work on adhesion by the various investigators indicates the better adhesion as measured by the various wash, swell tests, etc., of the MC for aggregate under even adverse conditions—that is, moisture, dust, lyophilic aggregate (1) (2) (14) (17) (32) (37) (43). Available information also shows higher stability of MC Mixes and that to get a suitable stability, workability, the proportions of the MC and filler, or gradation are not as critical. Powers (24) enumerates his preference for MC's as follows:

1. Definitely developes a stronger binder than SC;

2. It does not lose its workability to any extent under varying working conditions;

3. It develops sufficient binding strength to be successfully laid with lower filler content aggregate;

4. It is less susceptable to displacement by water;

5. Leaves a residual product of rather soft consistency. Wood (42), Nevitt (20), and Bollen (4), Russel (29), Nuss (21), Kelley (13), and others find that the proper grade of MC results (in addition to the reasons outlined above) in a tougher surface with better non-skid properties, less critical proportioning of oil and aggregate, better specification control of the asphalt present, and higher proportion of a penetration asphalt for a given viscosity or workability.

Powers points out that there are possible difficulties in reworking MC mixes as compared to SC, but feels that as MC is more easily proportioned and laid, the possibilities of reworking are smaller as long as the base is satisfactory. Benson (2) and Russell point out that under some conditions some MC materials harden in the surface and result in mats which may be brittle, harsh, and lack self-healing properties. Because SC materials do harden very slowly they have been recommended as a possible solution. However information on the relative merits of this solution is not available as an insufficient number of SC and MC mixtures from mats under the same conditions have been examined to conclusively indicate that excessive hardening happens irrespective of the type of MC type cutback or that it is limited to medium or light cutbacks. Another solution is the use of softer base MC's. This is to be recommended because the inherent advantages substantiated in both the field and laboratory of the MC's should not be lost. Kansas, South Dakota, and Illinois, have recently written suitable specifications in this respect. California has had a soft base MC, their FOMC (32), and for some time have found it a very successful material. Unfortunately with most of the current specifications, in spite of a 300 penetration maximum distillation residue, materials appreciably above 200 penetration cannot be shipped because of the 100 plus ductility requirement on the distillation residue. Typically asphaltic materials softer than 200 penetration are too soft, and ductility specimens flow apart instead of pulling to a thread. That this failure of the soft distillation residue is not a typical ductile failure is demonstrated by the 100 plus ductility of a soft non-ductile distillation residue air reduced to 100 penetration. Requiring a 100 plus ductility on distillation residue air reduced to 100 penetration would be a plausible requirement for soft

base MC's, permitting 200-300 penetration distillation residue on current distillation residue penetration limits or even softer if proper modifications are made to evaluate the consistency of the distillation residue; and the distillation is modified for the various grades. These soft base residues can be expected to reduce the brittleness of the mat as experience indicates that merely changing the penetration requirements 20 degrees from the 50-60 penetration asphalts has shown at least 50% reduction in cracking and similar.

The trend of softer asphalt cements for asphalt concrete, sheet asphalts, and similar, is also consistent with the available data (9) (25) (26) (27). Recent investigators, although probably differing in their exact limits feel certain that the final consistency and ductility of the asphalt cement in a bituminous surfacing must be over a given minimum. Bituminous material hardens to a measurable degree under atmospheric conditions and do so to a higher degree under the high temperatures of mixing and laying. The general recommendations summarized by Hubbard are: (1) to use as soft an asphalt cement as is possible without danger of shoving, moisture trouble, etc.; (2) use as high a per cent as is possible without shoving or displacement: i. e., increase the film thickness to the maximum practical limit thereby reducing the permeability as well as reducing the effect of air on the extensive asphalt films; (3) Mix and lay mixes at as low temperatures and in as short a time as is possible; (4) compress mixtures thoroughly and use seal coats to further reduce air permeability.

The use of the highest viscosity liquid asphaltic material is consistent with research. It has been demonstrated that the stability or cohesion as measured by the various stability and shear tests is higher with the higher consistencies (15) (32). Consequently the proportion of the mix, filler, aggregate are not as critical. The resulting thicker films tend to give a tough resilient mat more impervious to moisture, and weathering. Finally the adhesion of the bituminous material relative to water on aggregate is improved with the higher consistency material (14) (17) (32) (37).

The proper proportioning of bitumen in road surfaces involves numerous variables and many methods of proportioning such mixtures have been proposed. Some methods are based on surface area evaluated by means of formulae or charts in terms of grading surface area, type of filler, etc. Other methods are based on maximum density or minimum voids. Stability, flexibility, cohesion, and shear tests on laboratory mixes also are used to evaluate the proper bitumen content. In general although the analysis and probable application are rational the variables are so numerous and often large that the actual application may be empirical, and there is a trend towards abandoning the formulae and equations and relying on laboratory observations and judgment of the engineer. It is believed that probably laboratory proportioning of the bitumen and on this basis determining the proper constant for the formula which is then used to adjust the bitumen content for such field variations as occurs is most ideal.

The use of formulae to proportion the bitumen content in the low cost road as reviewed by Hubbard (10, Hveem (32) and Nevitt (19)

indicates that most successful application is on the basis of surface area. However there are other variables. The character of the aggregate, specific gravity, shape, texture, gradation, absorptive properties, and percentage of voids are also involved. In addition Nevitt found it necessary to include some consideration of the oil viscosity for a more general application of these formulae.

Bollen (3) (4) includes in his modified New Mexico formula a factor characterizing the type of fines on the basis of the Nebraska stability index "S". Such an index or the indices of Traxler (36), Newman (18), or Traxler, Omstead, and Bollen (38) could also be applied to modifications of other existing formulae to correct for variations in the type of fines.

Hubbard in summarizing the proportioning of bitumens points out that the proportion of bitumen in a hot mix hot aggregate requires application of entirely different concepts than for the low cost road cold mix proportioning, and that these hot mixes can be best proportioned on the basis of voids or densities. However data by Vokac (40) (41) indicate voids are not critical as long as they are within reason and accordingly the void concept should be only used to supplement proportions decided upon in the laboratory. The general experience indicates surface area considerations in the proportioning of hot mix surfacing will result in a lean mix. Similarly proportioning bitumen in a cold mix type of surfacing on the basis of voids would give an excessively rich mix.

Investigations of these discrepancies by Hubbard seems to indicate that a pore viscosity concept could be more generally applied to various types of mixes, but as yet enough information is not available for practical application.

Physical evaluation in the laboratory of the service behavior of bituminous mixes has progressed. Numerous tests have been developed. The shear tests (15) (26) (27), impact tests (25) (26) (27) (39) compressive strength tests (40) (41), beam deflection (25) (26) (27), Hubbard-Field Stability (11), Hveem (32), stabilometer, cohesiometer (32), and Pieffer-Cell (22) are a partial list of these tests. Prediction of the service behavior of the materials in use as indicated with these tests are being made, and possibly the most complete investigations, including tests on the same mixtures by most of the above tests, in this respect are those of Vokac (40) (41). In general he finds the impact test shows the least correlation with service behavior. On the basis of one year's performance the per cent voids is not critical so long as they are within reasonable limits. He does however believe that the excessive voids will eventually be deterimental. The Skidmore-Shear in this comparison is not found sensitive to factors offering satisfactory service behavior. Vokac feels direct shear is foreign to service conditions. The Hubbard-Field being a measure of the flow of the mix through an orifice involves shear and is unsatisfactory for the same reasons as the Skidmore-Shear tests. Further the dimensions of the orifice and mold must be so carefully controlled that it is difficult for different investigators to get good checks. Vokac does find that the compressive strength, the elastic limit, and the modulus of the elasticity, the latter being the least critical, are indicative of service behavior of the cold mix powdered asphalt flux bound surfaces and the hot mix

50-60 penetration asphalt surfacings. The limits for satisfactory behavior are outlined in Table II. The above does not necessarily mean that if the tests fall outside the prescribed limit for a B surface the mix will be a failure. This is true even when working with mixes and bitumens exactly duplicating those used to make the above comparisons. It is quite possible, as an examination of the Table will show, to have a mix exhibiting good characteristics outside of the prescribed limits for B mixes.

Raschig and Doyle (26) (27), and Rader (25) have evaluated the modulus of rupture and elasticity of sheet asphalts at low temperatures and correlated their results with the cracking of the pavement. Their investigations indicate that maximum bitumen content, maximum film thicknesses and maximum densities using softer asphalts are desirable.

The adhesion of bituminous materials to aggregates is receiving more and more consideration on the part of highway engineers as it is felt that good adhesion is necessary for satisfactory service. The fundamentals of this particularly difficult problem have been discussed by McLeod, winterkorn, Kelley, Tremper, Mack, Nicholson, Hveem, Lee and others (14) (17) (32) (27) (43). Higher adhesion is observed with increasing consistency of bitumen, with the medium and light cutbacks over SC, with high asphaltene content asphalt over low asphaltene content asphalt, with tars and typically positive high gravity material over typically negative spot materials. Further large differences in adhesion have been observed between aggregates, depending on both their physical and chemical nature, and upon their adsorbed films. Improvement of the adhesion of bitumen to aggregate is indicated possible by the proper choice of bitumen or aggregate or by treatment of the aggregate. Because of the complexity of the subject all investigators feel that the field conditions of the treatment of the aggregate and bitumen should always be adopted in the laboratory's investigation of the adhesion.

SEALS

The primary purpose of the seal is to prevent the passage of moisture or water from the top of the surface into the mat: secondly it should add strength to the surface as that is where the wear and tear is concentrated. Finally proper depth of penetration continued impermeability, reasonable life, ease of application, are required for the optimum sealing material. As discussed here the term sealing is limited to treatments whose primary purpose is to waterproof and strengthen the surface but not thicken it. Any cover used it only in small amounts and used only to protect the seal. Any procedure thickening the wearing course, although it serves the purpose of a seal, is here defined as an armor coat or similar.

Accepted practices and recommendations vary. RC, MC, SC, and soft asphalts of various consistencies are used in various localities, with probably RC being the most generally accepted. The quantities vary with the condition and type of surface and type of seal.

The most comprehensive survey of seals is the report of Legault (16). The relative merits of the various types of seals investigated are summarized in Chart I on percentage basis of the perfect seal. For complete details the original bulletin is recommended. Legault's investigation included

field studies of the relative moisture permeability, wearing surface qualities and bond, and the non-skid properties. Laboratory studies of the extent of penetration of the seal at various quantities applied under various conditions of the various seal materials, the rate of penetration, permeability, tensile strength, compressive strength, and drying time, were made. On the basis of these investigations the special seal whose properties are summarized in Table III has the best combination of properties, a very high rate of penetration was shown to insure the full seal, rapid drying facilitates application and does not appreciably hold up the traffic, its high strength and high ductility insure a complete, tough, leathery seal. Its high impermeability is shown and assures a good seal. Further it can be anticipated that this material will show relatively high adhesive properties because of its high consistency base (14) (17). Reports by Wyoming, Montana, and Iowa, where the seal has been successfully used verify Legault's observations. The use of SC, MC, and soft base materials as seals is not generally a good practice, and although the seal with the use of these seems satisfactory, better results can be secured by using materials particularly adapted for the purpose. It would appear that although there are variations in the surface, the requirements of the bituminous sealing materials for optimum results are more exacting than the requirements in the mat itself. Investigations of the practice of making lean mixes immediately followed by a seal as compared to making a normal binder content mat are not available. However the opinion of active technologists (9) (10) (12) 25) 27) (32), leaves one with the impression that it is undesirable to over-emphasize stability at the expense of other important properties which in this instance are flexibility, workability and permeability, and that lean mats with extremely thin bituminous films are quite subject to oxidation and other weathering deteriorating influences even if they are properly sealed. All other conditions being equal the optimum per cent bitumen should be able to better resist moisture, ravelling, impact, abrasion, and oxidation. Stanton and Hveem further point out that in the event of reworking, richer mixes are preferable (12) (32).

It would be of interest to lay two surfaces identical in every respect except for the binder content, and after a suitable period in service, compare the consistency of the asphalt present in the mat. The possibilities are the lean thin films would have hardened appreciably faster than the films in the normal bitumen content mix.

GENERAL

The more exact and less empirical of the physical and chemical properties of bituminous mixes: that is, strength and adhesion are rapidly permitting the more rational choice of a suitable bituminous surfacing material as to type, character, and quality of bitumen. There are so many variables and the development is so rapid that the exact universal limits are not possible. More data and a closer correlation between laboratory and field results will however make this possible. This should result in the rational design of bituminous surfaces on the same sound basis as bridge and other structural design.

Summarizing verification of accepted general practices as indicated by recent research data we find:

1. MC materials preferably positive spot, are the best primes with quantities and the viscosity depending upon the base and climatic conditions.

2. Light and medium cutback asphalts and soft penetration asphalts are more acceptable than SC for bituminous mats although preliminary data indicates hardening of MC's current up to a year ago. The proper specification of soft base MC materials should correct this difficulty. The proportioning of bituminous materials at present depends on the type of mix, the properties of the aggregate, etc., and is best determined in the laboratory or its equivalent. The hot mix can be approximately proportioned on the basis of voids; the cold mix on the basis of surface area, with both being more and more apportioned on the basis of laboratory tests of stability, etc. Adhesion should be considered in the proper choice of bituminous materials with the higher consistency asphalts, the cutbacks and the tar and typically positive spot materials showing better adhesion than soft asphalts, SC materials, and negative spot materials.

3. RC and particularly a special seal RC material seems the most suitable seal material in spite of the usual practice of using numerous other types. The proper per cent of binder in a mat with a subsequent seal also seems more desirable than a lean mat with an immediate seal.

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TABLE I

TABLES OF ASPHALTIC PRODUCTS ACCORDING TO AGGREGATE ASPHALT CEMENT

Penetration Macadam	85-100, 100-120, 120-150 penetration according to temperature—intensity of traffic.
Plant Mix (Cold Laid)	85-100 penetration with liquefier, or 250 to 350 without liquefier according to aggregate gradation.
Plant Mix (Hot Laid)	
(a) Sand Asphalt	70-85, 85-100 penetration.
(b) Sheet Asphalt	50-60, 70- 80 penetration
(c) Asphaltic Concrete	60-70, 70- 85 penetration

LIQUID ASPHALTIC PRODUCTS

(See Specifications Liquid Asphaltic Materials)

	RC Rapid Curing				MC Medium Curing					SC Slow Curing			
	1	2	3	4	1	2	3	4	5	1	2	3	4
Dust Layer										X			
Primer Closed Surface					X								
Open Surface						X							
Seal Coat Light	X					X	X						
Heavy		X	X	X									
Mulch Treatment						X					X		
Cold Patch Summer		X	X										
Winter							X						
Road Mix (A) Coarse Aggregate Type			s										
Road Mix (B) Open graded, roller compacted, max. dia. 1¼", high per cent passing 10 mesh, low 200 mesh							X						
Road Mix (C) Dense Graded. Traffic compacted, max. dia. 1½", high per cent passing 10 mesh													
Dry Climate, cool weather											X		
Dry Climate, hot weather												X	
Severe Climate						X							
Cold-Laid Plant Mix													
(a) Open Aggregate—no fines				X									
(b) Dense Graded, 1" max. dia., high passing 10 mesh, low 200 mesh, traffic compacted—Dry Climate													X
Severe Climate							X						
(c) Open graded 1" max., high per cent 10 mesh, low 200 mesh, roller compacted								X					
(d) Open graded, 1" max., high per cent passing ¼", no 200 mesh									X				

TABLE II

PHYSICAL PROPERTIES—SERVICE BEHAVIOR CLASSIFICATION

(From Data by Vokac)

Pavement	HOT MIX SHEET ASPHALT			COLD MIX SHEET ASPHALT		
Classification	A	B	C	A	B	C
Stability at 140° F.	2,400+	1100-3600	1,300–	1,560+	1100-2180	1,780–
Shear at 140° F.	41+	22-59	27–	19.2+	16.8-29.0	24.2–
Compressive Strength	1,000+	575-1225	600–	810+	580-925	600–
Elastic Limit	850+	525-1025	525–	550+	300-630	340–
Modulus of Elasticity	12,000+	8000-18000	8,500–	32,000+	22000-41000	35,500–
Modulus of Perm. Def.	4,750+	1750-5750	2,000–	3,200+	1750-5200	2,700–
Voids Per cent	2.6+	2.0-6.4	3.2–	3.25+	1.60-5.70	3.35–

NOTE: Service Classification by Letter after 1 year service
A—Harsh, dry, abrasive.
B—Normal, satisfactory.
C—Rich, shoving, rolling.

Asphalt Cements—

Hot Mix: Principally 50-60 penetration asphalt.
Cold Mix: Powdered Trinidasco and flux.

TABLE III

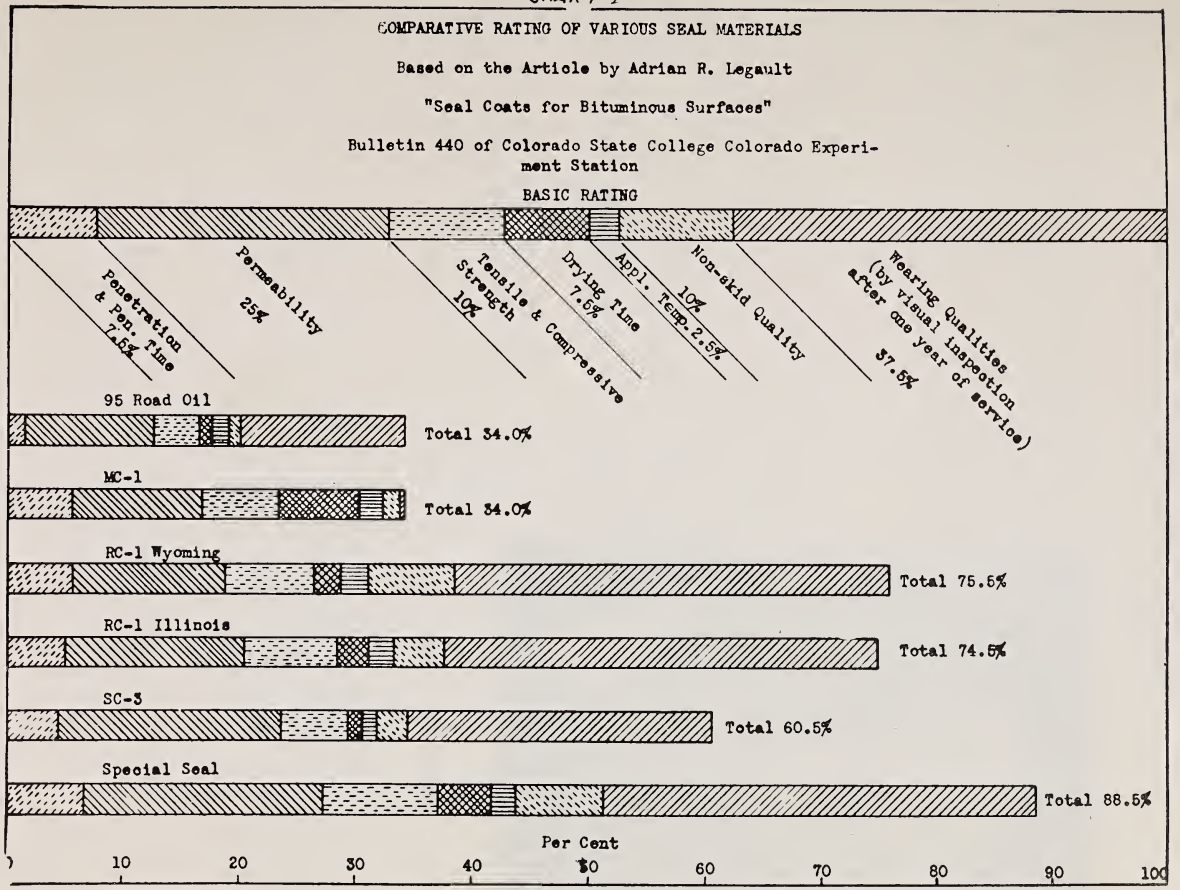
(Legault "Seal Coats for Bituminous Surfaces" Bulletin 440, Colorado Experiment Station).

RESULTS OF STANDARD TESTS OF BITUMINOUS SEALING MATERIALS

Type	Special Seal	RC-1	RC-1	MC-1	SC-3	95 Road Oil
Specification:	Manufacturer*	Asphalt Institute	Colorado	Colorado	Colorado	Colorado
Source						
Tests	Wyoming	Illinois	Wyoming	Wyoming	Wyoming	Wyoming
Viscosity Furol Secs.						
@ 77° F.	176			104		
@ 122° F.		132	92			
@ 140° F.				261		
Flash °F.	85	88	120	150	290	415
Distillation Per Cent off						
@ 374° F.	12.0	11.0	5.0			
@ 437° F.	25.0	21.0	17.0	2.5		
@ 600° F.	37.0	30.5	30.0	38.0	0.0	
@ 680° F.	39.5	32.5	32.5	42.0	2.0	
Float on original @ 122° F.						236
Penetration of Dist. Residue @ 77° F.	36	93	102	130		
Solubility in CCl_4						
Original					99.9	99.9
Dist. Res.	99.6	99.5	99.9	99.9		
100 Pen. Res.					68	97
Ductility @ 77° F. 5 cm/min						
100 Pen. Res.					110+	110+
Dist. Res.	110+	110+	110+	110+		
Ductility @ 39.2° F. 5 cm/min						
100 Pen. Res.					7.4	7.0
Dist. Res.	0		8.6	11.0		

*The specification on which this seal was furnished was developed from tentative specifications on which special seal was supplied to highway projects of several mid-western states.

CHART I


COMPARATIVE RATING OF VARIOUS SEAL MATERIALS
Based on the Article by Adrian R. Legault
"Seal Coats for Bituminous Surfaces"
Bulletin 440 of Colorado State College Colorado Experiment Station
BASIC RATING
Penetration & Pen. Time 7.5%
Permeability 25%
Tensile & Compressive Strength 10%
Drying Time 7.5%
Appl. Temp. 2.5%
Non-skid Quality 10%
Wearing Qualities (by visual inspection after one year of service) 37.5%
95 Road Oil
Total 34.0%
MC-1
Total 34.0%
RC-1 Wyoming
Total 75.5%
RC-1 Illinois
Total 74.5%
SC-3
Total 60.5%
Special Seal
Total 88.5%
Per Cent
0
10
20
30
40
50
60
70
80
90
100

A Critical Evaluation of Systems of Soil Classification as Applied to Highway Engineering

By

O. L. STOKSTAD, Soils Engineer
Michigan State Highway Department

Most literature dealing with the subject of soil investigations for highway purposes concerns itself with a discussion of frost heaving, drainage, fill construction, stabilization, and the various testing procedures with the computations involved.

Insufficient emphasis is being placed on the importance of getting acquainted in general with the various soils as they exist in the field and are encountered during highway construction operations. Test results are therefore, not sufficiently associated with individual soils, an association which is quite necessary if an endless repetition of sampling and testing is to be avoided.

The Argentine report to the Eighth Road Congress at the Hague—1938 is of special interest in this respect. The Argentine Republic is apparently developing a technique for making soil studies for highway purposes which combines the Pedological approach to soil classification with testing procedures designed to shed information as to the performance of various soil materials when used for highway subgrades. An interesting side light on the highway soil investigations in Argentine is that the soil surveys are also used in preparing a soil (edaphological) map of the Republic. The U. S. Bureau of Public Roads rendered assistance by loaning the Republic one of their soil specialists.

The purpose of soil classification is to simplify soil studies through organization of data, thus making it possible to cover more ground with less effort and thereby increasing the effectiveness of the soil engineering personnel.

The outline submitted by the chairman for the committee on Research requests a critical evaluation of systems of soil classification as applied to highway engineering. In this connection the discussion which follows will be confined to the subject of mineral soil investigations for road metal foundations and for general treatment of the highway right-of-ways.

The systems of classification suggested for discussion are as follows:

1. Classification of soil textures as illustrated by the triaxial chart

contained in Ohio Bulletin No. 85, "Some Effects of Soil, Water, and Maintenance of Highways." F. N. Eno.

2. The classification of soils into eight groups based on tests designed to reflect performance—the system developed and used by the U. S. Bureau of Public Roads.

3. The Pedological system of soil classification which separates soils into (1) zonal groups, (2) series (3) types and (4) phases. This system is in use by the U. S. Bureau of Chemistry and Soils and by soil scientists throughout the world.

At this point it may be well to differentiate between a soil and a soil material. The soil, as defined by Joffe, is a natural body, differentiated into horizons of mineral and organic constituents, usually unconsolidated and of variable depth. This body differs from the parent material below in morphology, biological characteristics, physical properties and constitution, chemical properties and composition. Soil materials, on the other hand, are the raw materials on which soil forming forces operate to develop a soil.

Methods for classifying soils may be either limited or general depending upon the capacity of the investigator and the use to which he expects to put the results of his investigations. He may classify (1) for the purpose of studying the influence of soils on some other subject, the soil itself being only of secondary importance in the study. He may classify (2) for the purpose of systematically cataloging all soil characteristics and thus shed light on the soil's influence on practical problems in any field where it may be a factor.

Pedalogists are interested in all soil characteristics and they operate on the premise that the results of their investigations will be of more practical use if they employ a system of classification which is based on characteristics of the soil rather than on the soil's influence on some other subject.

The study of soils is an ancient study and numerous methods of classifying have therefore been used. The following outline of classification methods is developed from material contained in publications by authorities like Wolfanger, Joffe, Kellogg, Hogentogler, and Housel.

I. Classification of soil materials.

A general study of the characteristics of soil materials without reference to the solum.

1. Texture
2. Surface area
3. Plasticity index
4. Shrinkage characteristics
5. pH values
6. Concentration of soluble salts
7. Silica—sesquioxide ratio

II. Classification based on, or influenced by, the character of the solum.

1. Utilitarian—A study of the soil's usefulness for various purposes.
    a. Production of timber
    b. Production of farm crops
2. Genetic—A study of factors instrumental in producing the soil.
    a. Botanic
    b. Biological
    c. Climatic
    d. Geological
    e. Geographic
    f. Topographic
    g. Lime (age)
3. Attributive—A study of the soil profile considering characteristics which have resulted from the influence of various genetic factors. "The soil itself, as a product of soil-building forces, is the object of classification"—Kellogg.
    a. Zonal groups
    b. Series
    c. Types
    d. Phases

III. Classification based on selected physical properties.

1. Bearing value
2. Stability—cohesion and internal friction
3. Shear value
4. Percolation rates

The scheme of classification now accepted and used in the science of pedology is attributive in character. The unit of study is the soil profile—a careful analysis of which, together with an observation of condition under which it exists, will yield information relative to factors used as a basis for all other systems of classification. This statement becomes especially true if laboratory test results are associated with the soil types from which the samples were collected.

Classification on the basis of soil textures alone without reference to other means of identification have limited usefulness. Other characteristics and conditions, as for instance drainage and uniformity, are ignored even though they may be as important as texture.

As previously mentioned the main purpose of classification is to provide soil information without necessitating continuous sampling and testing. The important need for classification no longer exists in so far as a particular problem is concerned if its solution is provided by laboratory test results. For this reason the usefulness of the eight groups (A1 to A8) as developed by the Bureau of Public Roads is definitely limited. This grouping emphasizes the laboratory approach to the field of soil engineering and the credit for the fine development of this approach belongs principally to Mr. Hogentogler and his co-workers.

It does seem necessary, however, to point out the need for a more comprehensive method of classifying soil in the field approach to soil engineering problems. The previously mentioned eight groups do not lend themselves to the recording of detailed field information without a large amount of field work and voluminous special notes.

Mr. Hogentogler has suggested verbally to the writer that the individual State Highway Departments use the A1 to A8 grouping in a general way and at the same time develop a more detailed system to fit its more extensive needs. For this purpose the Pedological scheme of soil classification offers excellent possibilities. This system is used and understood by pedologists in all countries and is simple enough so that an experienced highway engineer can become quite proficient at mapping in less than 3 months time. In addition there is a tremendous amount of soil survey information available for use by the highway engineer as soon as he is sufficiently familiar with the mapping legend to use it. The amount and distribution of this information is illustrated in Fig. 1 (obtained from the Bureau of Chemistry and Soils).

There are 3 arguments used by engineers in discrediting the pedological approach to soil classification for highway purposes. (1). The system is too complicated (2) the soils of some states have been insufficiently identified (3) the classification is for Agricultural application only.

With respect to the first criticism, Michigan has had no difficulty in training experienced highway engineers in the art of soil mapping. In fact the enthusiasm with which they have grasped the new tools of foundation studies has been responsible for the rapid development of the technique. The second criticism mentioned can be solved by working out a cooperative arrangement between the Highway Department, the U. S. Bureau of Chemistry and Soils, and the Soils Department of the local University or College whereby an adequate legend can be prepared within a relatively short time. The third objection is due to a misunderstanding. It is true that the agriculturist has made more general use of the soil survey than any other profession. The pedologist's interest is in the entire field of soils, however, and he tries to present his information so that it can be of service in any field where soils may be an influencing factor. The engineer need not feel, therefore, that his professional standing is jeopardized by adopting a pedological scheme of soil classification.

It is obvious that in any study of soils the serviceability of the study as well as operating efficiently must be influenced by the type of classification tools used. It is important therefore, in investigating

soils for highway purposes that a system of classification should be sufficiently broad to permit its utilization throughout the field of highway research, design, construction and maintenance.

It is important in a continuous highway soil program to use methods of soil mapping which permit the automatic recording of a large amount of detailed information without requiring time consuming laboratory work. Laboratory tests are necessary for supplying special information on some specific problem, but the bulk of all necessary design data should be supplied by the soil survey as inventory presented by the field worker.

The Michigan State Highway Department has found that the pedological scheme of soil classification provides an excellent means of studying soils for general highway purposes and that it serves as a foundation for all other soil studies in the Department. The soil type, in addition to serving as a unit in mapping, also serves as a unit to which laboratory test results, road behavior observations, construction problems and maintenance experiences are related. In other words, the soil type serves as a means for classifying and coordinating the accumulated soil experience of the entire highway department personnel.

Until quite recently the common impression has been that "the highway engineer is interested only in those soil properties upon which the serviceability of the pavement depends." (7)* It is now quite generally agreed that the highway engineer is interested in the entire highway right of way. This wider interest requires consideration of such subjects as landscaping, erosion control, shoulder stabilization, and to some extent, land use. The intelligent and efficient handling of these subjects, especially by the design division, requires a method for classifying the soil which reflects a large amount of detailed information about each individual soil.

In connection with this wider usefulness to which a soil survey can be put it may be of interest to discuss briefly the technique used in Michigan.

Pedological soil surveys in this State were started for research purposes in 1925 and the first survey was made for design purposes in 1927 by Dr. C. E. Kellogg, now Principal Soil Scientist, Bureau of Chemistry and Soils.

Mapping for design purposes did not become a routine matter, however, until after 1929. Since then the technique of making soil surveys and using soil information has developed rapidly.

At first only the more important projects on the primary system were studied. Costs were therefore, not a dominating consideration. Since soil studies have broadened, however, to include all highway projects on both primary and secondary systems it has become important that a small personnel function quickly and economically.

For purposes of soil mapping the great soil groups are divided into soil series, which have similar profiles, drainage, topography, and parent material. These series are usually given geographic names taken from

*Refers to publication listed under references.

the locality where they were first recognized. Typical Michigan series names are: Miami, Onaway, Ontonagon, Hillsdale, etc. Each of these series may vary in texture of surface material, and these textural classes are designated as loamy, sand, sandy loams, silt, clays, etc. The series name together with the class designation, gives us the soil type, which is the unit of soil classification. This is also the unit as previously mentioned to which all other soil information is related. Pedologists have found that the soil type may be used as a basis for the classification of physical and chemical data. When used in this manner it provides a very useful tool in the study of soils for highway purposes. This system of classification is recognized internationally, and the important literature in soil science is therefore based on it. The Michigan State Highway Department, in adopting this method of studying soils, was immediately able to take advantage of the soil survey information available in the form of published county soil survey reports. In addition it was also possible to profit by contact with the personnel of the Soils Department of Michigan State College.

During the history of an individual project soil knowledge is first used in connection with its planning. This involves the soil's influence on the highway location and on the type of road to be built. If county soil maps are available in the area, much can be learned through their study as to problems to be encountered on various alternate locations. In connection with secondary highway programs particularly, land use information will be of special importance in determining the advisability by road building activities of encouraging or discouraging agricultural utilization of a given area.

It is in the design of the highway that a comprehensive soil survey can be of real service to the highway engineer. In order to assist the design engineer in the use of soil survey information a subgrade design chart (Fig 2) has been prepared in which a few of the important design questions are answered in tabulated form for the various soils. This chart is sufficiently simple to make explanation unnecessary. It may be of interest, however, to note the variation in the recommendations for the Blue Lake sand and the Eastport sand—two soils of the same texture. This chart has increased the usefulness of the soil surveys tremendously in so far as the average designer is concerned and there is no reason why similar charts cannot also be worked up for other highway functions. It is in connection with design, however, that an intimate knowledge of soil conditions can be of greatest service. To further illustrate the use to which a soil survey is put by the Michigan State Highway Department, an attempt has been made in Fig. 3 to present graphically the mental picture of foundation conditions created by the soil classification survey shown at the top of the sheet.

After the plans are finished and the construction operations started purely soil activities become relatively simple. It then becomes necessary for the soil engineer to study the soil materials in the grade in order that frost heaving textures are identified and removed, and that a uniformly textured subgrade is prepared for the metal. Drainage conditions of the subgrade must be studied to determine the need for, as well as the type and location of special drainage structures. The soil

materials to be used for various purposes are checked to make sure that they are suitable for the use intended.

In all his activities the foundation on which the soil engineer stands is the soil survey. The classification of the soil, which it involves, is the first step in becoming acquainted with the various soil engineering problems to be encountered on a new project. An acquaintanceship with each soil personality is the principal means whereby the soil engineer can profit by past experience in dealing with soil problems presented by new projects.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF CHEMISTRY AND SOILS, H. G. KNIGHT, CHIEF

AREAS COVERED BY THE SOIL SURVEY

(JANUARY 1938)

100 50 0 100 200 300 M.

NUMBERED LIST OF RECONNAISSANCE SURVEYS

CALIFORNIA
1 Sacramento Valley, 1913
2 San Francisco Bay Region, 1914
3 Lower San Joaquin Valley, 1915
4 San Diego Region, 1915
5 Middle San Joaquin Valley, 1916
6 Central Southern, 1917
7 Upper San Joaquin Valley, 1917

KANSAS
8 Western, 1910

MICHIGAN
9 Ontonagon County, 1921

MINNESOTA
10 Lake of the Woods, 1926
11 Red River Valley Area, 1933

MISSOURI AND ARKANSAS
12 Ozark Region, 1911

MONTANA
13 Northern Plains, 1929
14 Central, 1934

NEBRASKA
15 Western, 1911

NORTH DAKOTA
16 Western, 1908

OHIO
17 Entire State, 1912

PENNSYLVANIA
19 Northwestern, 1908
20 Southwestern, 1909
21 South Central, 1910
22 Northeastern, 1911
23 Southeastern, 1912

SOUTH DAKOTA
24 Western, 1909

TEXAS
25 South, 1909
26 Panhandle Region, 1910
27 Central Gulf Coast, 1910
28 Southwest, 1911
29 South Central, 1913
30 Northwest, 1919
31 West Central, 1922
32 Trans Pecos, 1928

VERMONT
33 Entire State, 1930

WASHINGTON
34 Eastern Puget Sound Basin, 1909
35 Western Puget Sound Basin, 1910
36 Southwestern, 1911
37 Columbia Basin, 1929

WISCONSIN
38 Northeastern, 1913
39 North Part of North Central, 1914
40 South Part of North Central, 1915

WYOMING
41 Johnson County, 1931

LEGEND

DETAILED SURVEYS

Published: Before 1907; Since 1907

Completed but not published

In progress

RECONNAISSANCE SURVEYS

8

(See opposite corner for numbered list of projects)

HAWAIIAN ISLANDS

CANAL ZONE

PHILIPPINE ISLANDS

ALASKA

VIRGIN ISLANDS

PUERTO RICO

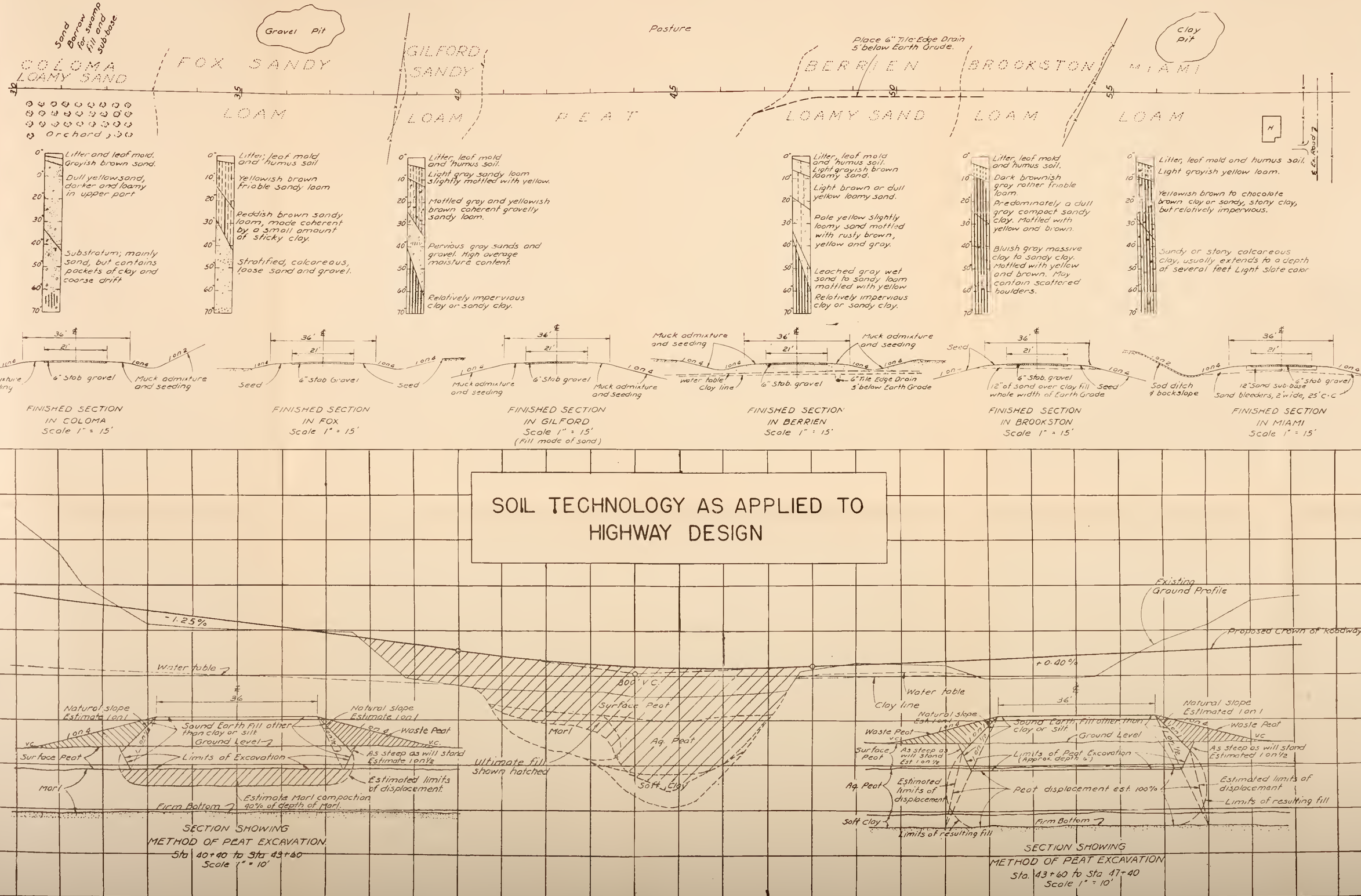
MICHIGAN STATE HIGHWAY DEPARTMENT
Murray D. Van Wagoner
State Highway Commissioner

FIG. 2.

SUBGRADE DESIGN RECOMMENDATIONS

SOIL SERIES	DESCRIPTION	Average Depth to Water Table	TREATMENT: Locate Grade at Any Elevation in Soil Profile	TREATMENT: Recommended Height of Grade Above Nat. Ground	TREATMENT: Sand Sub-Base Recommended	TREATMENT: Sub-Grade Mat Required for Gravel (See Specifications)	TREATMENT: Frost Heave Excavation	TREATMENT: Estimated Lineal Feet Per 100 ft. of Cut: Cut, Tile Edge Drain	TREATMENT: Construct Fills in 1 Ft. Layers	TREATMENT: Shoulder Stabilization Recommended	TREATMENT: Erosion Control for Slopes Suggested: Wind	TREATMENT: Erosion Control for Slopes Suggested: Water	BORROW: Available	BORROW: Per Cent of Shrinkage	BORROW: Suitability for Fills: Core and Swamp	BORROW: Suitability for Fills: High (Proctor Tech.)	BORROW: Suitability for Fills: Shallow	BORROW: Stabilization on Sandy Shoulders	BORROW: Top Soil	REMARKS	SOIL SERIES
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18		
Adolph	Poorly drained fine sand and silt over clay	0.5 to 1.0	No	2.0 to 3.0									No	15 to 25	No	Yes	Fair	Poor	Good		
Alger	Wind deposited very fine sand and silt over bedrock	Indefinite	Remarks	Remarks	No	No	300	100	Yes	No	Seeding only	No	Yes	10 to 20	Poor	No	Fair	No	Poor	(2-3) Bedrock may vary from 3' to 10'.	Adolph
Allendale	2' to 3' of poorly drained sand over clay	1.0 to 2.0	No	1.5 to 2.5	Remarks		300	600	Yes		Sod	No	No	15 to 25	No	No	Sand Good	Clay Good	Poor	(4) Sub-base desirable for portions of grade in clay	Alger
Alpena	Well drained gravel ridges	6.0 to 20.0	Yes		No	No	0	0	No	No	No	No	Yes	5 to 10	Excellent	No	Excellent	No	No	Possible source of gravel. Check for loose rock excavation.	Allendale
Amasa	2' to 3' of fine sandy loam over sandy drift (stony)	Deep	Yes		No	No	200	200	No	No	Admixture	No	Yes	10 to 20	Good	No	Good	Poor	Poor		Alpena
Antrim	Imperfectly drained loamy sand and gravel	3.0 to 6.0	Remarks	Remarks	No	No	0	500	No	No	Seeding	No	Yes	5 to 15	Excellent	No	Excellent	No	No	(2-3) Possible source of gravel. Establish grade 4' above water table.	Amasa
Arenac	3' to 5' of imperfectly drained sand over clay	3.0 to 5.0	No	0.0 to 1.0	Remarks	Yes	100	400	Yes	Yes	No	Admixture	Yes	5 to 15	Sand Excel.	No	Sand Excel	Clay Good	No	(4) Sub-base desirable for portion of grade in clay.	Antrim
Autrain	Well drained sand	Deep	Yes		No	Yes	0	0	No	Yes	Admixture	Admixture	Yes	5 to 15	Excellent	No	Excellent	No	No		Arenac
Baraga	Moderately well drained loam with bedrock at 4' to 8'	Indefinite	Remarks	Remarks	No	No	200	300	Yes	No	Sod	No	Yes	10 to 20	Poor	Yes	Good	Good	Fair	(2-3) Grade controlled by bedrock of irregular profile.	Autrain
Barker	Silty clay uplands	Indefinite	Yes		Yes	No	400	300	Yes	No	Sod	No	Yes	20 to 30	No	Yes	Fair	Excellent	Good		Baraga
Bellefontaine	Well drained sand and gravel	Deep	Yes		No	No	200	300	No	No	Admixture	No	Yes	5 to 15	Good	No	Good	No	No	Possible source of gravel.	Barker
Bergland	Poorly drained clay	Shallow	No	2.0 to 3.0									No	20 to 30	No	Yes	Poor	Excellent	Excellent		Bellefontaine
Berrien	3' to 5' of imperfectly drained sand over clay	3.0 to 5.0	No	0.0 to 1.0	Remarks	Yes	100	400	Yes	Yes	No	Admixture	Yes	5 to 15	Sand Excel.	No	Sand Excel.	Clay Good	No	(4) Sub-base desirable for portions of grade in clay.	Bergland
Blue Lake	Well drained sand gently rolling	Deep	Yes		No	Yes	200	300	No	Yes	Remarks	Plant	Yes	5 to 15	Excellent	No	Excellent	No	No	(10) Admixture for deep cuts.	Berrien
Bohemian	Lake deposits of very fine sand and silt	Indefinite	Yes		Yes	No	800	500	Yes	No	Sod	No	Yes	15 to 25	Poor	Yes	Remarks	No	Poor	(16) Suitable if placed 3' below flat earth grade.	Blue Lake
Brady	Imperfectly drained sandy loam	3.0 to 5.0	Remarks	Remarks	No	No	0	500	No	No	Seeding	No	Yes	10 to 20	Good	No	Good	No	Fair	(2-3) Establish grade 4' above water table.	Bohemian
Bridgman	Sand dunes	Deep	Yes		No	Yes	0	0	No	Yes	No	Remarks	Yes	5 to 15	Excellent	No	Excellent	No	No	(11) Admixtures and planting.	Brady
Brimley	4' to 6' of imperfectly drained very fine sand over clay	3.0 to 5.0	No	0.0 to 1.0	Yes	No	800	500	Yes	No	Seeding	No	No	15 to 25	Remarks	Yes	Remarks	Good	Fair	(14-16) Suitable if placed 2' below flat earth grade.	Bridgman
Brookston	Poorly drained clay	Shallow	No	2.0 to 3.0									No		No	Yes	Poor	Excellent	Excellent	Excavate heavy sod before filling.	Brimley
Bronson	6' to 10' of sandy loam over clay	Deep	Yes		No	No	200	200	No	No	Sod	No	Yes	10 to 20	Sand Excel.	No	Excellent	No	No		Brookston
Bruce	Poorly drained fine sand and silt over clay	0.5 to 1.0	No	2.0 to 3.0									No	15 to 25	No	Yes	Poor	Poor	Good		Bronson
Brule	Wet stream bottom soil	Remarks	No	Remarks									No	20 to 30	No	Yes	Poor	Good	Good	(1-3) Subject to overflow.	Bruce
Buck	1' to 2' of sandy loam over clay	Indefinite	Yes		Remarks	No	200	400	Yes	No	Sod	No	Yes	15 to 25	No	Yes	Fair	Good	Poor	(4) Sub-base desirable for portion of grade in clay.	Brule
Burt	Poorly drained very fine sand and silt over bedrock	1.0 to 2.0	No	2.0 to 3.0									No	15 to 25	No	Yes	Poor	No	Fair		Buck
Channing	Imperfectly drained fine sandy loam over clay	2.0 to 3.0	No	1.0 to 2.0									No	15 to 25	No	Yes	Fair	Fair	Poor		Burt
Chatham	Well drained stony loam	Deep	Yes		No	No	100	300	No	No	No	No	Yes	5 to 15	Good	No	Excellent	No	No	Loose rock abundant.	Channing
Clyde	Poorly drained clay	Shallow	No	2.5 to 3.5									No	20 to 30	No	Yes	Poor	Excellent	Excellent	Remove heavy sod before filling.	Chatham
Coldwater	Imperfectly drained clay	Indefinite	Yes	Remarks	Desirable	No	200	200	Yes	No	Sod	No	Yes	20 to 30	No	Yes	Poor	Excellent	Excellent	High grade line desirable.	Clyde
Coloma	Well drained sand rolling	Deep	Yes		No	Yes	200	300	No	Yes	Admixture	Admixture	Yes	5 to 15	Excellent	No	Excellent	No	No		Coldwater
Colwood	Poorly drained very fine sand over clay	0.5 to 1.0	No	2.5 to 3.5									No	20 to 30	No	Yes	Fair	No	Good		Coloma
Conover	Imperfectly drained clay	Indefinite	Yes	Remarks	Desirable	No	200	200	Yes	No	Sod	No	No	20 to 30	No	Yes	Poor	Excellent	Excellent	High grade line desirable.	Colwood
Coventry	Well drained loam over sand and gravel	Deep	Yes		No	Likely	100	200	No	Likely	No	Planting	Yes	5 to 15	Good	No	Excellent	No	No		Conover
Crosby	Poorly drained clay	Shallow	No	2.0 to 3.0									No	20 to 30	No	Yes	Poor	Excellent	Excellent	High grade line desirable.	Coventry
Crystal Falls	0' to 4' of loam over igneous bedrock	Indefinite	Remarks	Remarks	No	No	0	200	Yes	No	No	No	No	10 to 20	No	Yes	Fair	Poor	Fair	(2-3) Grade controlled by bedrock of irregular profile.	Crosby
Detour	4' to 10' of stony loam over limestone bedrock	Indefinite	Remarks	Remarks	No	No	200	300	Yes	No	No	No	Yes	10 to 20	Fair	No	Good	Poor	Fair	(2-3) Grade controlled by bedrock.	Crystal Falls
Diana	3' to 8' of poorly drained stony loam over igneous rock	0.5 to 1.0	No	3.0									No	15 to 25	No	Yes	Fair	Poor	Poor	(2-3) Grade controlled by rock and water table.	Detour
Eastport	Sandy beaches and lake borders	4.0 to 10.0	Remarks	Remarks	No	Yes	0	100	No	Yes	No	Admixture	Yes	5 to 15	Excellent	No	Excellent	No	No	(2-3) Establish grade 4' above water table.	Diana
Echo	Sandy wash from adjacent uplands	Indefinite	No	Remarks	No	No	200	400	Yes	No	Seeding	No	No	10 to 20	Poor	Yes	Good	No	No	(3) Cuts possible but unlikely due to position.	Eastport
Emmert	Stony sandy loam	Deep	Yes		No	No	200	200	No	No	No	No	Yes	5 to 15	Excellent	No	Excellent	No	No	Possible source of gravel.	Echo
Emmet	Well drained loamy sand rolling to hilly	Deep	Yes		No	Likely	200	200	No	Likely	Planting	Remarks	Yes	5 to 15	Excellent	No	Excellent	No	No	(11) Admixtures for more sandy phases.	Emmert

SOIL TECHNOLOGY AS APPLIED TO HIGHWAY DESIGN


Sand Borrow for swamp fill and sub-base
Gravel Pit
Pasture
Place 6" Tile Edge Drain 5' below Earth Grade.
Clay Pit
COLOMA LOAMY SAND
FOX SANDY LOAM
GILFORD SANDY LOAM
PEAT
BERRIEN LOAMY SAND
BROOKSTON LOAM
MIAMI LOAM
Orchard
Litter and leaf mold. Grayish brown sand.
Dull yellow sand, darker and loamy in upper part
Substratum; mainly sand, but contains pockets of clay and coarse drift
Litter; leaf mold and humus soil
Yellowish brown friable sandy loam
Reddish brown sandy loam, made coherent by a small amount of sticky clay.
Stratified, calcareous, loose sand and gravel.
Litter, leaf mold and humus soil.
Light gray sandy loam slightly mottled with yellow.
Mottled gray and yellowish brown coherent gravelly sandy loam.
Pervious gray sands and gravel. High average moisture content.
Relatively impervious clay or sandy clay.
Litter, leaf mold and humus soil. Light grayish brown loamy sand.
Light brown or dull yellow loamy sand.
Pale yellow slightly loamy sand mottled with rusty brown, yellow and gray.
Leached gray wet sand to sandy loam mottled with yellow
Relatively impervious clay or sandy clay.
Litter, leaf mold and humus soil.
Dark brownish gray rather friable loam.
Predominately a dull gray compact sandy clay. Mottled with yellow and brown.
Bluish gray massive clay to sandy clay. Mottled with yellow and brown. May contain scattered boulders.
Litter, leaf mold and humus soil.
Light grayish yellow loam.
Yellowish brown to chocolate brown clay or sandy, stony clay, but relatively impervious.
Sandy or stony calcareous clay, usually extends to a depth of several feet Light slate color
5 Ex. Road
36'
21'
Muck admixture and seeding
6" Stab gravel
Seed
Water table
Clay line
6" Tile Edge Drain 5' below Earth Grade
12" of sand over clay fill whole width of Earth Grade
Sod ditch & backslope
12" Sand sub-base
Sand bleeders, 2' wide, 25' c-c
FINISHED SECTION IN COLOMA Scale 1" = 15'
FINISHED SECTION IN FOX Scale 1" = 15'
FINISHED SECTION IN GILFORD Scale 1" = 15' (Fill made of sand)
FINISHED SECTION IN BERRIEN Scale 1" = 15'
FINISHED SECTION IN BROOKSTON Scale 1" = 15'
FINISHED SECTION IN MIAMI Scale 1" = 15'

-1.25%
+0.40%
Water table
Existing Ground Profile
Proposed Crown of Roadway
Natural slope Estimate 1 on 1
Sound Earth Fill other than clay or silt
Ground Level
Limits of Excavation
As steep as will stand Estimate 1 on 1/2
Waste Peat
Surface Peat
Marl
Aq. Peat
Soft Clay
Ultimate fill shown hatched
Estimated limits of displacement
Estimate Marl compaction 40% of depth of Marl.
Firm Bottom
Clay line
Limits of resulting fill
Peat displacement est. 100%
SECTION SHOWING METHOD OF PEAT EXCAVATION Sta 40+40 to Sta 43+60 Scale 1" = 10'
SECTION SHOWING METHOD OF PEAT EXCAVATION Sta. 43+60 to Sta 47+40 Scale 1" = 10'

SECTION II

Fundamentals of Bituminous Construction

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State Highway and Public Works Commission
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Report of the Committee on Fundamentals of Bituminous Construction

By

THOS. E. STANTON, Jr., Materials and Research Engineer
California Division of Highways
Sacramento, California

Report Presented by

J. W. POWERS, Materials Engineer
Arizona State Highway Commission
Phoenix, Arizona

The question has been asked: What new purpose is served by the Montana National Bituminous Conference which has not been, or cannot be served by existing organizations such as the Asphalt Institute and the A. A. S. H. O.?

While much of the same ground might be and often is covered in meetings and in published papers by other organizations, nevertheless

it is true that no other conference has thus far covered as thoroughly those bituminous problems that are peculiarly western.

The scope of the work of this committee is contained in the definition of its duties. Mr. A. H. Benedict,* Chairman of the Fundamentals Section in 1937, states that the Fundamentals Section should be confined to "statements of proved principles and their application to bituminous design"; also, "The subject of fundamentals is not bounded by any sharp line of definition, but rather is the filler in the sandwich between research and construction."

It is specifically implied in the above definition that there must be contact or liaison between those responsible for research and those responsible for construction and maintenance of highways. There is a tendency for most national organizations to be limited or handicapped in their effectiveness because of the necessity for representing widely different viewpoints and local conditions. Many such meetings have been confined to technologists or administrators and the atmosphere is often rather rarefied for the men who must actually supervise construction or maintenance work.

While all information is valuable, and much light on local problems may be obtained from experience in far distant regions, in the final analysis the building of a specific highway is largely a local and personal problem of some few individual engineers.

A great deal of valuable work has been done and is still being done by technologists and researchers in numerous laboratories throughout the country, but it must be realized that their efforts bear no fruit until they are actually applied on the job by some construction or maintenance engineer. And conversely, the laboratory man's efforts may be misspent and the findings of the researcher unimportant unless he is in close touch with the problems of the field engineer. It is in this bringing together of the research, design, construction and maintenance engineers that the Montana Bituminous Conference has been most outstanding.

Whether or not this conference continues to be the best medium to carry on this work, we owe a considerable debt of gratitude to Mr. McKinnon and his associates for taking the initiative and sponsoring the conferences, as there is a real need for a strong organization to exchange experiences and compile data on bituminous road surfacings in the West.

There is no thought to belittle or disparage the value of information and procedures originating in any area, and the valuable work done by national conferences is too well recognized to require defense. Nevertheless, more intensive study of local problems is usually carried on by regional than by national groups, which must cover a widely divergent field. In other words, it appears necessary that we have both regional and national conferences. Each type of organization can serve an important need. Two or even three conferences would probably be justified in appropriate divisions of the United States and Canada.

As an example of the mutual stimulation and interchange of ideas which can arise in a conference attended by both "theoretical" and

*Technologist, The Asphalt Institute, San Francisco, California

"practical" men, out of the discussions at the 1937 conference at Glacier National Park came the cue for one important theme discussed this year, namely, the increased trend in recent years toward brittleness and excessive cracking of asphaltic concrete pavements. Mr. Emil Skarin, of Edmonton, Canada, comparing, to their disadvantage, the pavements built in recent years with those constructed twenty or twenty-five years ago, stated at the 1937 conference that he was "often ashamed of the recent work," and asked the assembled technical men why the modern pavements so often showed distress from cracking and apparent undue brittleness of the asphalt. As part of the answer to Mr. Skarin's question, we have this year an outstanding paper by Mr. C. L. McKesson, Director of Engineering and Research for the American Bitumuls Company, entitled "Durability of Asphaltic Binders."

The lack of durability of many asphaltic pavements is beyond question a most serious problem, and is being attacked vigorously by many state laboratories. Among these might be mentioned particularly the states of Michigan, Minnesota, Kansas, and Arizona which have contributed much valuable data.

The Materials and Research Department of the California Division of Highways has likewise been engaged in an intensive study of the subject during the last two or three years. In this study it has had the very active assistance of the technical staffs of the asphalt producers in the state. The results of these studies, which are still under way, will be presented at some future conference as a further contribution on this subject.

Another hardy perennial among problems which plague the bituminous engineer is the much discussed hydrophilic aggregate, with the early deterioration and heavy maintenance costs which result when it is used in bituminous pavements. We now have the spotlight thrown on another contributing factor, namely the oil or asphalt which has poor adhesion to aggregates and may be largely responsible for failures due to water action. While these offending oils or asphalts are not yet branded with some distinguishing term such as hydrophilic or hydrophobic, nevertheless the existence of variables among the asphaltic binders is demonstrated in a paper entitled "Role of the Swell Test in the Selection of Materials and Design of Dense Graded Bituminous Mixtures" by J. W. Powers, Engineer of Materials, and W. G. O'Harra, Chemist, for the Arizona State Highway Department.

FUNDAMENTALS OF BITUMINOUS CONSTRUCTION

In addition to the definition by Mr. Benedict, quoted previously, Mr. D. L. Cheney, of the Montana Highway Department defines the work of the section as being intended to cover the design of sub-bases and bases, and factors influencing the selection of aggregates and asphaltic material, whereas it is intended that the work of the Construction Section is to be devoted to actual construction. As it appears to be an almost unanimous opinion that the Fundamentals discussion should cover the entire field of subgrade, base, and surface, our presentation, therefore, will be confined as far as possible to "statements of proved principles and their application to bituminous design" insofar as they appear to relate to the subgrade, base, and surface of highways.

SUBGRADES AND FOUNDATIONS

There appears to have been little change in thought during the past year as to the general principles of soil analysis and subgrade construction. The practice is becoming more universal to compact soils at optimum moisture content and to compare the compaction actually obtained with some standard method. This comparison requires that field men have training in methods of test. Mr. J. O. Martineau, of Quebec, makes the following statement: "Any efforts by road building organizations to improve subgrade stability should first be directed towards the training of specialists for both field and laboratory work, so that investigation of soil conditions may be made adequately and proper remedies applied where needed." Mr. Swanberg states, "Minnesota has had such an organization for several years, and considers it indispensable for the obtaining of adequate and economical design."

The interdependence of sub-base, base and surface in the design of bituminous pavements is ably discussed in a paper by Professor W. J. Emmons. Professor Emmons prefaces his remarks with the statement that all traffic loads are carried by the subgrade. The apparently marvelous performance of certain roads of light and cheap construction is still in accordance with the principles of mechanics. A state of equilibrium exists in any road resisting the effects of traffic, and the function of the various factors is susceptible to analysis. Published articles by Hawthorn and Housel along this line have attracted a great deal of attention. Housel's analysis apparently has been substantiated by field tests by the Michigan State Highway Department.

Mr. Martineau's discussion seems to indicate that stabilization and treatment of roadbed and subgrades is one phase of highway engineering which more than any other is not so much in need of further investigation or research as in the application of knowledge already at hand; in other words, the general principles governing the construction of good foundations and subgrades are well understood. What is required is that this knowledge be actually applied in the field by the engineers who are responsible for the design and construction of highways.

The consolidation of soils at optimum moisture was pioneered in California approximately ten years ago, and has been standard procedure since 1929. Some states have been inclined to classify soils on the basis of liquid limit, pasticity index, and shrinkage determinations, but in view of the fact that these determinations are made on the material finer than 40-mesh, and the soil actually used in construction may contain a relatively small percentage of such fine particles, California prefers to make determinations on the material as it will be used, except that particles coarser than 1" are screened out before testing in order that results may be less erratic. Regardless of test methods, however, the intent is the same; namely, the rejection, if practicable, of the least suitable material, and the efficient consolidation at optimum moisture of the material finally selected for subgrade or base course.

Professor Emmons stresses a point which certainly must be recognized by the highway engineer, that is that the soil survey is destined to become as much a matter of routine as the line survey. With more precise

methods of design, surfacing thicknesses may be varied with the subgrade supporting power. In any event, the application of reasonable safety factors should result in definite reduction in maintenance costs.

Soil profile plans are being used more widely, and are an essential requirement for intelligent selection and disposal of materials. Swanberg states, "Minnesota goes in for extensive soil selection to place the most suitable material in the upper portion of the road, and to excavate to a depth of 2 feet or more known objectionable frose heave soils." Rejection of adverse soils and the substitution of better material can, of course, be practiced only if better material is available. When it is impracticable to replace adverse soils in the upper layer of the roadway, the practice of stabilizing such soils with some agent such as emulsified asphalt, portland cement, chemicals, or other means, is becoming increasingly popular, as the principles and methods of application are better understood.

The possibility of using some admixture for stabilizing local adverse soil has attracted the attention of many technicians, and a great deal of knowledge has been accumulated concerning the factors which govern the adhesion of asphalt films to mineral aggregates, as well as the effect of base exchange capacity of soils, and similar relationships. Much remains to be done in this field to devise methods which are at once effective, economical, and feasible under construction conditions. We all know of excellent results secured with mixtures of even the lightest grades of SC oils with aggregates which have greater affinity for organic liquids than for water, and there are instances where exceptionally durable pavements and surfaces built with SC-4 oil have withstood heavy traffic for a number of years. However, when the local soils possess adverse characteristics which render them unsuitable for oil treatment, then the need for alternate types of construction is very definite.

SELECTION OF BITUMINOUS MATERIALS

A great deal has been written in discussion of the proper grades of bituminous materials to suit variable subgrade and traffic conditions, and there are available many publications by The Asphalt Institute and others giving in considerable detail the purposes for which the various products have been found suitable. This subject was also covered by Mr. Buchanan in his review for the Fundamentals Section last year. It is not believed necessary to cover this ground again.

It may be of interest, however, to note some of the trends which have become apparent within the last year or so. Foremost is the trend toward softer paving asphalts, and so far as all evidence indicates, there is nothing to discourage this move. On the contrary, the mounting evidence of widespread distress in asphaltic pavements due to brittleness, makes it imperative that every possible step should be taken to reduce this type of failure. In his discussion on "Durability of Asphaltic Binders" Mr. McKesson makes several significant statements, among others, "Soft asphalts have longer useful life than hard asphalts and may be used without sacrificing stability." McKesson also states, "Ductility is the most important physical property of asphalt or asphaltic residuum recovered from pavements." The data submitted and points covered by

him seem to leave little justification for any opposition to the use of soft asphalts. It appears that there may be, however, some question whether ductility as now measured is the most significant property of asphalt.

Mr. Powers agrees that ductility tests as made at present are not satisfactory. At this point it may be appropriate to attempt a diagnosis of the actual significance of the ductility measurement. Mr. A. R. Ebberts has offered the suggestion that ductility is a measure of the thoroughness or degree of dispersion of the colloidal system composing the asphalt. It is well known that by means of slight cracking the ductility of an asphalt may be increased measurably, but is does not necessarily follow that this treatment improves the quality of an asphalt.

McKesson has furnished a great deal of data in tabular form showing the effect of various influences on the consistency of asphalt. Swanberg offers some contradictory evidence and states that he is inclined to doubt "the softening of an asphalt by means of volatile, as in a cutback, will ultimately result in an asphalt of less ductility." It may be stated here that the observations of cutback roads in California appear to indicate that with the standard MC grades, at least, there is an undue tendency toward hardening and brittleness. For this reason, California is using medium curing cutback made with SC-6 road oil as a base. (This is the approximate equivalent of 300 penetration asphalt). Results so far appear to be definitely more satisfactory.

Powers offers some additional very interesting data to substantiate his contention that hardening of asphalt is chiefly a surface phenomenon.

He states in commenting on McKesson's paper:

> "As these experiments indicate that the hardening of asphalt is more probably a surface reaction, the depth of which is unknown, and is not appreciably reversible, is it not possible that in thin films of asphalt, such as exist in asphaltic mixes, the oxidation has proceeded completely through the film and produced a permanent change which is not reversible? If such as the case re-heating of asphalt mixes before testing should not appreciably affect results.
>
> "This discussion leads to a problem which warrants more thorough investigation, the problem being—to what extent are the properties of asphalts existing in thin films in asphaltic mixes affected by recovery by extraction methods or by re-heating?"

In any event, it appears that most investigators are in general agreement that the essential test data may be summed up as follows. Required: Accurate measures of (1) consistency, (2) durability, and (3) adhesion.

(1) Better tests for consistency are needed. This property is now measured by three rather unsatisfactory methods; namely, penetration float viscosity, and the Viscosimeter. It would be much more satisfactory if viscosity were expressed in asbolute units and all products, whether very fluid or comparatively hard, expressed in the same scale.

(2) It is necessary to know whether or not an asphalt will remain in its original condition or to what degree and how rapidly it will harden under weather and traffic.

(3) Adhesive qualities of asphalt as determined by its ability to resist displacement by water from mineral aggregate should be measured. There are no tests in general use at present which are entirely satisfactory for this purpose.

In effect, all we need to know is whether a bituminous material is of the proper consistency to meet construction requirements, whether it will remain soft enough during the life of the pavement to prevent cracking and undesirable abrasion, and whether it will adhere to the mineral aggregate in the presence of moisture. The adoption of better consistency tests will undoubtedly be delayed, but it is not unlikely that either through oxidation tests on the asphalt or by accelerated weathering tests on standard mixtures of asphalt and aggregate, it will soon be possible to rule out in advance all asphaltic products which lack durability in service.

Tests to determine the hydrophilic or organophilic properties of aggregates have been in use for several years, and have been the subject of much discussion and intensive study of late. As a member of the group on Fundamentals, Mr. J. W. Powers has submitted new data on the Swell Test, showing the effect of road oils from various sources. Mr. Powers concludes, "If the Swell Test is to be used as a basis for acceptance of mineral aggregate, the characteristic of the oil used in such testing will have to be specified," and "The results . . . indicate that the service behavior of questionable mineral aggregates may be modified by certain oils."

Powers' data also shows the consistently superior adhesion developed by cracked materials compared to the uncracked asphalts. This fact has been demonstrated a number of times and indicates that the asphaltenes are undoubtedly most effective in promoting adhesion. The belief held by some engineers that asphaltenes are inert is not substantiated by data available to the writer.

Mr. Powers deserves to be complimented not only on the excellent paper, but also in view of the fact that the Swell Test was first developed and reported by him in 1930, it indicates an admirable scientific attitude that he should be the first to point out the shortcomings of his own brainchild.

The variation shown by Powers is not entirely unexpected, however. Several years ago a brief investigation along similar lines was made in the California Laboratory using a very high swell aggregate sampled from the road along the northeast shore of the Salton sea in the Imperial Valley. The Swell Test was measured using various California oils with a resulting range of swell from .019" to .584". The oils showing the lowest swell contained a small percentage of paraffin wax, and those showing the highest swell contained sulphur. While it has not been proved that these two ingredients were responsible for the different behavior of the oils in question, the possible effect of paraffin or sulphur should not be overlooked.

Mr. Russell reports that they have also found that different material showed a different amount of swell with different grades of oil.

Powers' suggestion that the oil used in testing should have a Xylene equivalent of 30% or less in the modified Oliensis test is worth serious consideration, and the suggestion is offered herewith that at least throughout the Western States, an oil of standard characteristics should be adopted for use in the Laboratory for swell and stripping tests. It is further suggested that the grade of oil might preferably be SC-4, which grade can be used for either road or plant mix construction, and in the laboratory in both the swell or wash-off and stripping tests.

With some exceptions, there appears to be a trend toward the increased use of slow curing oils in preference to the cutback asphalts, although Swanberg reports that Minnesota used the MC cutbacks in their open graded mixtures. Russell of Wyoming states that SC-3 and SC-4 are favored because of their low cost and satisfactory performance. Rapid curing cutbacks RC-1 and 2 are used extensively for seals. Russell states that "in light of Wyoming experiences . . . I am . . . convinced that the Minnesota type of grading could be successfully used with residual oils of SC-3 or SC-4 grade."

In the Southeast the sand asphalt mixture is made largely with naptha cutback such as RC-1 and 2.

SELECTION OF TYPE OF MIXTURES

Dense graded mixtures versus open-graded types.

In this paper describing the open-graded type of bituminous construction, Swanberg of Minnesota refers to nomenclature suggested by Mr. J. E. Buchanan at the last Conference. It is the feeling of the Chairman that this nomenclature should be repeated herewith, as too much support cannot be given to any move toward standardized terminology. According to Mr. Buchanan, the definitions are as follows:

Aggregates

Coarse Aggregate:	Aggregate containing particles larger than ¼ inch.
Fine Aggregate:	Aggregate containing particles smaller than ¼ inch.
Macadam Aggregate:	A coarse aggregate consisting of fragments of substantially one size or a limited range of sizes the smallest of which is ¼ inch.
Graded Aggregate:	Aggregate containing proportions of particles of all sizes from the maximum size to dust.
Open-Graded Aggregate:	A graded aggregate with a limited proportion of particles passing the No. 10 sieve and not more than 5 per cent passing No. 200 sieve.
Dense-Graded Aggregate:	A graded aggregate with a continuous representation of all particle sizes from the maximum size to dust, and containing between 5 and 15 per cent passing No. 200 sieve.

Bituminous Surfaces

Surface Treatment: Any bituminous surface which has a final thickness less than one inch.

Pavement: Any bituminous surface which has a final thickness greater than one inch.

The above suggestions for a standard terminology appear to be very suitable, although one or two minor modifications might be an improvement. The first two definitions would be better as follows:

Coarse Aggregate: Aggregate consisting of particles larger than 4-mesh.

Fine Aggregate: Aggregate consisting of particles smaller than 4-mesh.

It is realized, of course, that many aggregate gradings will overlap two or more of the above definitions. Nevertheless, for the purposes of simplification, descriptions similar to these should be adopted and universally agreed upon.

Minnesota practice, as described by Swanberg, furnishes a satisfying example of a highway engineering department utilizing local materials in the most economical and in the simplest manner. Glacial gravel deposits in Minnesota are numerous and are distributed throughout the State, and as the existing gravel contains little dust, the gradation closely corresponds to that described above as open-graded type. For this construction, cutback asphalt of MC-2 or 3 grade for road mix, and MC-4 or 5 for plant mix, has been considered most suitable. The extent to which the most available materials are utilized without expensive processing is excellent evidence of the extent to which real engineering is being applied to the highway problem. Further examples of this adaptation of methods to local conditions are the so-called multiple lift (penetration macadam) constructions of Oregon, and the sand asphalt pavements of the southeastern states.

It may be well at this point to sound some warning against the dangers of standardization if carried beyond its proper scope. For greater economy and efficient results, the selection of bituminous binder and the type of construction should be adapted to the local conditions of available aggregates, traffic, and climate. In states which have a wide range of materials and conditions, many types of treatment and construction may be necessary in the several regions. It sometimes happens that the urge toward standardization directly conflicts with the greater efficiency of specialization.

Mr. Russell, of Wyoming, describes Wyoming experience, which has tended to the use of the dense graded type of mixture, with which they are finding that slow curing oils such as SC-3 and 4 are proving to be eminently satisfactory and economical. He cites evidence to show that greater width of oil mats and increased shoulder width have materially reduced maintenance costs.

While there is some difference of opinion on the relative merits of open graded and dense graded mixtures, the fact that satisfactory surfaces have been built with both seems to make it unnecessary to advocate the use of either type without regard to local conditions. Most Eastern states appear to use open graded mixtures, whereas in the Southwest dense graded types predominate, probably for the reason that local materials in desert areas usually contain considerable fines and meet dense graded requirements without difficulty.

Swanberg has given excellent reasons for the use of open graded types in Minnesota; and the well known penetration macadam roads in Oregon are undoubtedly the most suitable means for utilizing crushed rock from the extensive lava flows in that state.

Mr. W. H. Mills, Jr., of South Carolina, has presented an interesting description of another type of mixture which utilizes existing sand present in large areas in the Southeast. He reports many miles of this construction covering several states, and furnishes considerable detail as to gradation of sand, percentage of asphalt, etc. Mixtures are made both in plants and on the road with traveling mixers. Mills comments that "machine mixes containing the same quantity of asphalt are richer in appearance than road mixes." This same tendency caused a great deal of trouble in California when plant mixes were first used, following the earlier experience with road mixing. The quantities of oil were largely fixed by "judgment" based on the appearance of a fresh mix and the result was that most of the early plant mix jobs were too dry and suffered considerable raveling. This is cited as proof that it is not always safe to depend on the eye or personal judgment of even "experienced" field men.

Mills' statement that moisture up to 6 or 8% in the average sand is not objectionable and may even be beneficial is rather startling to some of us in the West who have been limiting moisture contents to 1½%. It would be interesting to know how much moisture the sand asphalt mixes actually contain when they are finally consolidated.

Swanberg comments, "Mr. Mills discusses the greater stability of mixes in which the sand carries from 15 to 20% of clay. The greater stability can probably be attributed to the better grading of the aggregate from stability standpoint; an increase in the consistency of the asphalt by the presence of filler and possibly absorption of the volatile fractions of the cutback by the clay."

Mills' paper should encourage Western engineers to look with a designing eye on some of the sand dunes and beach sands which are available in certain regions.

Fahnestock, of North Carolina, gives further description of the sand asphalt mixtures, with special reference, however, to the so-called drag retreatment process in which a rather fine dense graded aggregate is used to level off and resurface old pavements, the material being mixed in place on the road by means of a special wire broom drag. The apparent economy of this treatment and the very favorable reports of the final result which have been brought back by western engineers, makes this process well worth looking into.

Swanberg reports similar work in Minnesota, and considers that the quantity of asphalt reported by Fahnestock appears excessive and likely to contribute to 'fat spots,' judged by Minnesota experience.

Powers states that treatments of a similar nature have been carried on in Arizona, utilizing in every case a medium curing product with aggregate graded as flolows:

Passing ½" square opening	100%
Retained on 3-mesh sieve	20-30%
Passing 10-mesh sieve	40-50%
Passing 200-mesh sieve	7-12%

This mixture is placed over old badly cracked pavements to a depth of approximately 1 inch. Powers states:

> "While this treatment has been very effective as far as smoothing up the old pavement, it has not been very effective in preventing cracks from the old pavement showing through into the new resurfacing.
>
> "In order to overcome and prevent cracking of the newly laid resurfacing, we have in many cases where grade line would permit, placed an intermediate layer of approximately 2 inches of suitable and stable natural material between the old pavement and the new. The maximum size allowed in the intermediate layer of natural material is usually ½ inch with rock content (above No. 3 sieve) ranging between 25% and 50%, depending upon the source.
>
> "While this type of construction has been done for probably not over two years, there is to date no evidence of cracking or checking of the new surface even though the original pavement may have been badly cracked."

After the bituminous pavement has been constructed, whether dense graded, open graded, or macadam type, it usually gets a seal coat sooner or later, whether it is needed or not. The place of seal coats has been discussed by Ernest H. Coffin, Supervisor of Wayne County Highway Department, Richmond, Indiana. Mr. Coffin places emphasis on the use of small screenings or coarse sand for seal coats. The following comment on his paper is made by Russeli of Wyoming:

> "Mr. Coffin in his paper has indeed stressed a most important fundamental of bituminous surfaces.
>
> "Soon after we started the construction of bituminous surfaces we realized this necessity and called for a plain seal of RC-1 on all contracts. Repeated sealing on dense graded mats soon leads to slippery surfaces and the only alternative then is a cover of either stone chips or sand.
>
> "If the surface is thoroughly sealed, however, and covered with chips all of which pass ⅜" or ½" with 0-5 pass No. 10 the mat should be adequately protected against both air and water. An additional cover of sand would improve riding conditions

and make a more non-skid surface. I would however suggest the sand seal as a reseal after the original chip seal has had a few years of service and is ready for an additional coating.

"We are at present calling for stone chip seals on many of our new projects and maintenance forces are using sand as a cover on most of our resealing."

Factors influencing the selection of seal coat types are discussed in a paper by Earl B. Lockridge of Indiana. Mr. Lockridge has presented an excellent detailed account of the theory and practice of seal coats, and he emphasizes one point which is not always kept in mind when the type of seal is being selected. He states that seal coats may be applied for different reasons. It might clarify the situation somewhat if distinct terminology could be used to differentiate between a treatment designed to prevent the entrance of water into the pavement, which is of course correctly designated as a seal, as compared to treatments applied to prevent a dry surface from raveling. or to reduce slipperiness. The methods and technique may often be varied to advantage where the intent is not definitely to provide a seal against moisture. Lockridge emphasizes the fact that both asphalts and aggregates have been condemned at times because they had not been properly proportioned. In his opinion, it is an error to attempt to produce a pebbly surface with coarse texture rather than the so-called sandpaper finish. A great many people who object to traveling over the "roaring retreads" will agree with him.

Lockridge does not touch on one factor influencing seal coat construction which has been of much interest in California; namely, the danger of concentrating moisture beneath a pavement due to the prevention of evaporation. Most bituminous mixtures, even of the dense graded type, will permit the passage of water vapor at a sufficient rate to prevent an excessive accumulation in the upper layer of the subgrade. In cases where the upward movement of water is appreciable, a seal coat may cause moisture to accumulate immediately under a bituminous surface to such an extent that the bearing power of the subgrade is reduced below the safety point.

While it is true that on new construction any material in the upper layer of the roadbed should have adequate supporting power when saturated, nevertheless, there are many old roads and places where funds do not permit the reconstruction of a poor subgrade. In such cases surfacing methods should be avoided which facilitate the accumulation of moisture.

There have been a number of cases where roughness and breaking up of oil roads can be traced to this cause. This does not mean, in any case, that seal coats should be abandoned in regions where there is heavy rainfall or severe winter conditions, but it does mean that local conditions should be studied before application of a seal coat.

In addition to the papers involving more or less proved principles and their application to bituminous design, we have a paper by Mr. N. M. Finkbiner. Engineer of Materials, Oregon State Highway Department, describing an investigation on the significance of the compressive strength of asphaltic concrete mortars. Mr. Finkbiner's paper should be read in its entirety to appreciate the significance of the data submitted.

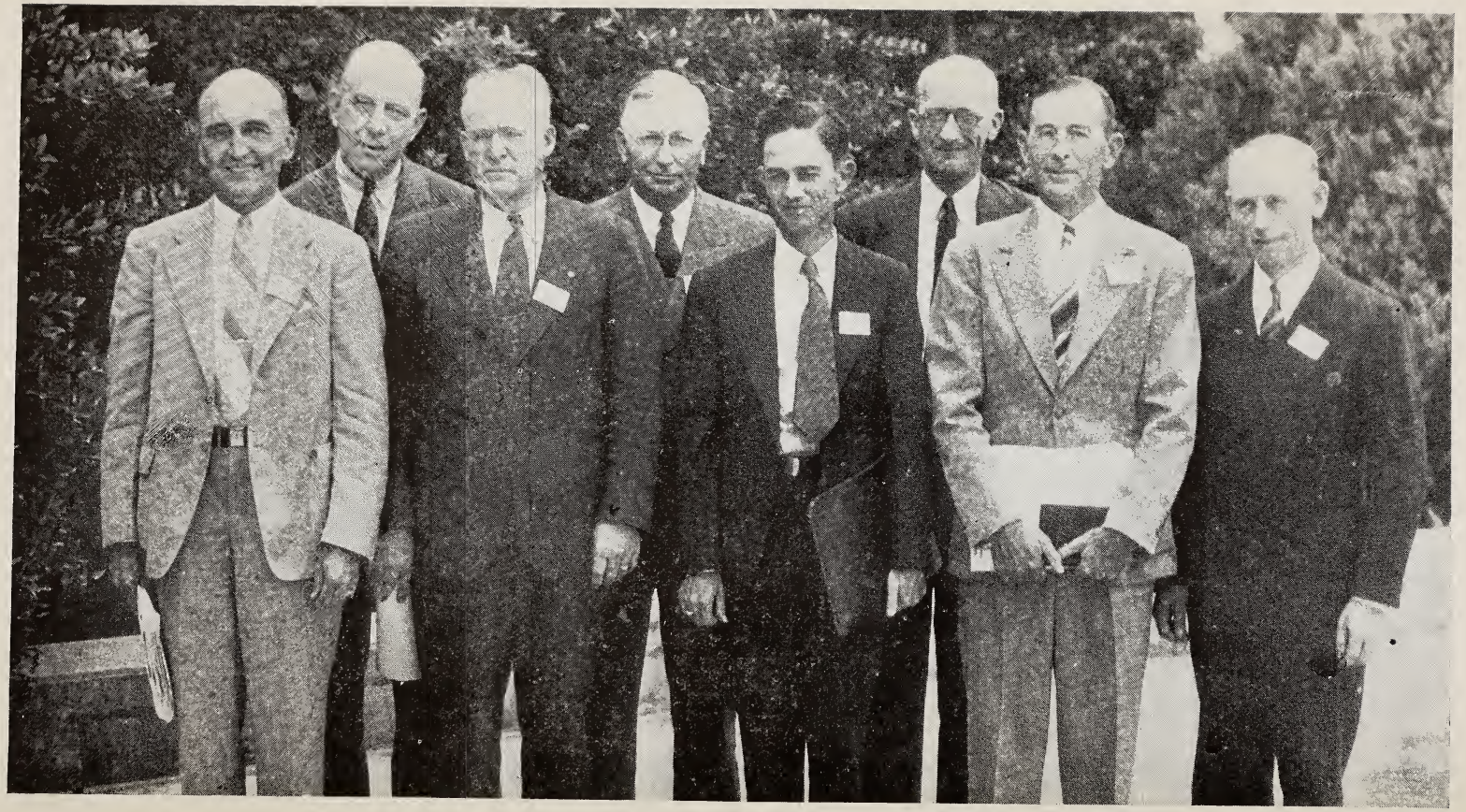
He submits data to show the effect of variations of the amount of asphalt, amount of filler, and the amount of mortar on the compressive strength of sand asphalt mixtures. Test results are plotted on a triaxial chart to form an isodynamiagram in order to indicate the interrelationship in percentage of sand, filler, and asphalt to produce compressive strengths higher than 600 pounds per square inch.

Finkbiner summarizes his observations as follows:

1. Variation of the percentage of mineral filler in the mixtures studied did not have as great an effect upon the compressive strength of these mixtures as a variation of the percentage of asphaltic cement.

2. A change in the value of the asphaltic cement, mineral filler ratio did not affect the compressive strength of the mixtures as much as a change of the value of the asphaltic cement, sand ratio.

In conclusion, I desire to express a debt of gratitude to Mr. F. N. Hveem, Senior Physical Testing Engineer, California Division of Highways, who assisted materially with the analyses and critical comments on the papers covered by this report.



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Mr. Powers: Gentlemen. As you know from the proceedings of yesterday and the summary that has been read this morning, there is somewhat of an overlapping in the Research Section and the Fundamentals Section, so from my point of view if there are any discussions that you would like to continue from yesterday I don't believe they will be misplaced at this session.

Mr. Stanton, in writing his review, divided it into three different sections. The first section is "Subgrades and Foundations," and at the present time we will discuss that particular phase of it.

Since I happen to be one of those who has written a paper and also have to act as chairman, it seems to me that I should be privileged to ask a question or two myself, so for that particular reason I am going to ask Professor Housel to give us some data, if he will, from his work which would indicate to us, (and I am sure you will be interested in it), what thicknesses of base course are necessary on certain types of soil. Professor Housel:

Prof. Housel: Michigan Mr. Chairman and Gentlemen: That question seems to be one that we were discussing yesterday. I will be glad to enlarge on it to whatever extent I can, and to do so would like to make use of the blackboard.

In the first place, the most definite answer to Mr. Powers' question as to thickness of base required on different types of subgrade would be that in order to determine that particular factor we necessarily must make some physical tests of the subgrade soil in order to know what its capacity to sustain loads will be. For cohesive soils, clay soils as most of us know to be the least resistant, the shearing resistance which we discussed would be a necessary factor to be determined before the definite answer could be given as to the thickness required. Our clay subgrades vary in bearing capacity through the year due to the seasonal fluctuation in moisture content.

Mr. Powers showed me earlier this morning one sort of empirical index that he has been using in connection with such soils in which, for different ranges of lineal shrinkage of subgrade soils different thicknesses of base course were required. As an empirical index I think that that is a logical thing, because we know that in clay soils their volume changes are a measure of the amount of moisture that can be absorbed, and that along with the increase in moisture content, measured in that way, there is a decrease in shearing resistance. Therefore, as a substitute for what you might call an actual measured shearing resistance that is a logical method of approach.

What we need, however, is more shearing resistance tests at these varying moisture contents. Then we may

be able to get a closer correlation between moisture content and shearing resistance. In the early work that Eno did he made some actual determinations over a period of, I believe, about two years, that showed that seasonal fluctuation in moisture content to a maximum, as you would expect, in the spring of the year, or during the period from the late winter to the spring. In general I think that it is fair to us to assume that in the clay soils they are liable to come pretty close to their maximum capacity to absorb moisture, so that the shrinkage criterion would be a useful one.

Another factor which of course must be considered is the ability of the base course to distribute loads. Referring to the sketch that is still on the board, the load applied to the base is distributed laterally depending upon the shearing resistance of the base, and relieving the clay subgrade of the concentration of pressure immediately under the load. We have very few determinations of the shearing resistance that is actually developed by these mixtures. Mr. Powers has a shear cylinder, described in an article which many of you have seen, and we now have built in the Michigan laboratory a similar shear cylinder. We are attempting in the laboratory at this time to duplicate on laboratory samples of the field mixture the densities that were measured in the field in connection with the load test that we discussed. Just as soon as we determine that we can compact these mixtures to these field densities, and if we can measure a shearing resistance by that test which is comparable to the determinations that were made in the field by the load test, we will then be able to effectively design the base before it is built.

Now, of course, with load tests, if we measure the shear after it is built it is too late to design it. The problem of devising the proper laboratory test looks to be well on the way to solution.

There is one thing that I did not mention yesterday in connection with the load tests that rather surprised all of us who were more or less expecting to find that the subgrade finally carries the load. Professor Emmons made that statement, and I think it is one to which we all agree, but there is a little complication in that conception which was brought out by these load tests.

We expect that as the load is placed on the surface the first thing that will happen is that the concentration of pressure will develop under the load, and the subgrade will be the first thing that starts to fail. We would expect that it would take considerable sagging of the surface to develop the shearing around the boundaries of the loaded area. Considering that we have two stress reactions or resistances, first the supporting pressure of

the subgrade and then the distributing action of the base, we would expect the sequence to be that the subgrade would fail first and then punching shear through the surface would be developed.

Out in the field we were surprised to find that the sequence of those reactions was reversed in practically all of our tests. In other words, the rigidity of the base (that is a 6-inch gravel base and a 3-inch oil aggregate on top of that, or an equivalent of a 9-inch base) with respect to the subgrade seemed to be sufficient, so that instead of following the subgrade, instead of sagging, the base distributed the load laterally before it transmitted it to the central column.

In other words, what I have tried to indicate is that the load was carried out to the side first and not right under the load. The pressure at low settlements was very much reduced and it wasn't until the load had been built up to a point where shearing of the base started that we began to get the high concentration immediately under the load.

The sequence in which the two stress reactions developed suggests that these bases which are more rigid than the subgrade are not going to be deformed at the same rate as the subgrade. Therefore, before we develop subgrade bearing capacity we will have at least started to shear through the surface. It appears that we can scarcely afford to shear through the surface, because that means the break-down of that structural element, the slab, so it may be that we will have to design the slabs to carry the load in shear more or less independent of the subgrade bearing capacity.

In other words, what I am trying to point out is that possibly we are going to be faced in some cases with floating the slab on a subgrade which may have a very low bearing capacity, just like a concentrated load on a sheet of ice. That brings beam action into the problem, and indicates the necessity of either making the base flexible enough to deform at the same rate as the subgrade or rigid enough to transmit the entire load to the subgrade in shear without a concentrated supporting pressure under the central column.

Frankly, the status of those tests and the analysis and finding that the sequence of those reactions was just the opposite from what we expected left us a little bit confused with regard to design. I am not prepared to say until we have completed our laboratory studies, whether we should recommend a thickness of base sufficient that the punching shear through the base will carry the load, or whether we should try to specify the thickness to make the base more flexible so it will follow the subgrade deformation as it takes place.

I will repeat what I said yesterday, in an attempt to give a more specific answer to Mr. Powers' question. From these tests and our general experience we are of the impression that the present more or less standard 6-inch to 9-inch bases that are used conventionally over most any type of soil are thicker, perhaps considerably thicker, than are necessary over rigid high bearing capacity subgrades, such as the Fox profile that Mr. Stokstad spoke of yesterday, and I am pretty definitely of the impression that these same more or less conventional thicknesses are quite inadequate over many of the clay subgrades which actually do take on high moisture contents and suffer the loss in shearing resistance.

Does that, Mr. Powers, cover your question?

Mr. Powers: That covers about what I expected. Thank you. Since Professor Housel has referred to the empirical method we use for determining depth of base course, it might be interesting to give you those particular data. They are based on the shrinkage of the minus 40 materials.

When it is from 0 to 2% we use no base course materials or no overlying materials. When the linear shrinkage is from 2 to 4% we use 6 inches; 4 to 6%, 9 inches; 6 to 9%, 12 inches; and above 9%, 15 inches or more, and it is not unusual for us to place 18 or even 24 inches.

If it is possible, I would like to have a discussion of that particular phase from the floor, or comments or questions that may be answered by some of the Conference Board.

H. G. Nevitt: Missouri Professor Housel made some remarks on design which happen to fit in with some observations I made in the field, without having his theory, and I think it is such a large and important subject that I would like to make a few remarks.

His results, that is, his observations in the field, indicate that beam action may come into effect. There have been three theories of design, I believe, generally speaking. One is the punching shear, of which I believe Professor Housel is the leading exponent. Bernard Gray leans toward the distribution of stress over the subgrade area by a spreading action; and then we have the beam action, which I believe I have urged more than anybody else, although of course all of these have been discussed widely by a number of people.

Professor Housel's conclusions seem to indicate that beam action comes into play, and he brings up the point that we might have too rigid a foundation. I believe I

have observed that very thing in the field in driving over the large territory that I try to visit occasionally. I believe that I have noticed a tendency toward what I call "over-design" of the base. We are tending towards heavy bases which are beginning to get rather rigid. As a result we begin to get a breakdown which is quite different from what we used to have. We are building rather thin mats on top of very rigid bases and now we are beginning to get cracking due to this rigidity. There now is a tendency to go to too heavy bases, which gives you this rigid construction, and to too thin mats. I believe some of our older mats and some of our older construction have stood up where they had an adequate base yet not too thick a base, and an adequate mat, one of 2½ to 3" in thickness, which is perhaps the answer to some of these points. Professor Housel's remarks rather fit into such a theory.

I wonder if we cannot keep the flexibility of our surfacing by balancing the design of the mat, which is stronger in the beam phase, against the gravel foundation, which is less strong in bending although it may be quite strong from the punching shear standpoint. I wonder whether good design may not lead us to mats which are a little thicker than some of the rather light mats we have been heading toward, and to bases which are not quite so heavy, so we keep our roads to the idea which has always been behind bituminous construction, that we have a flexible type of surface. In other words, I think that if we over-design the bases it is an economic disadvantage due to the extra expense. If we do not over-design the bases and do not under-design the mats, we may hold ourselves to the original idea of a bituminous surface, which is a flexible surface able to adapt itself to the subgrade and to pass the stress on to the subgrade and utilize the subgrade bearing power to the maximum extent.

These remarks seem to lead up to or generally point toward the possibility of that conclusion. I don't say that they indicate it, but I just wonder if the points that have been made do not indicate that maybe we should stay to a little bit thicker mat and not go to too heavy bases, yet of course go to adequate bases. Very seldom have we had too heavy bases in the past. I want to rush in to say that. Our bases have been generally too light, but maybe we can go to the other extreme and make them too heavy and rigid.

Mr. Powers: Thank you, Mr. Nevitt. Is there anyone else?

A. E. Stoddard: Iowa

As an exponent of a state which has been carrying on the other extreme, I feel a cosmic urge to give our opinion. We build lots of base with very thin mat. There are a couple of points in Mr. Housel's discussion which it seems could still be enlarged upon. One is that the determination of the lineal shrinkage is a definite thing insofar as laboratory determination is concerned. It will be an easy matter to classify soils upon that. The determination of the sheer is dependent upon the moisture content, and to know whether you have a sample which exactly represents the maximum moisture content to which the soil will be subjected is an investigation which takes considerable time.

If you are in a hurry, as most laboratories are with their work, the basis upon which you can accomplish the end with the least work is the one which is most desirable. I believe that if we work out the relationship between the shears for the various classes of soils and get that down to some other determination which could be made more rapidly, it would be a big help in the classification from the point of view of determining the amount of base necessary.

We build about five inches of stabilized base on most of our soils. That is an over-design in a good many cases, and the amount of mat we put on is about ¾-inch thick. That gives a perfectly good wearing surface, and if the base is adequate that mat is adequate.

The ultimate end of any of these surfaces is to produce a riding surface which is satisfactory, and it has been our experience that when you had one that was flexible enough to follow the contours of the soil you had an unsatisfactory riding surface.

Mr. Powers: Thank you, Mr. Stoddard.

Dr. Hans Winterkorn: Missouri

Mr. Chairman, it might be in order to remind the audience at this moment that the shearing resistance of the soil is dependent not only upon the moisture content but also upon the structure, which may change while the soil specimens are being prepared and tested. If you make your test on the disturbed soils, they may after a while acquire a structure and their stability may increase. Besides that, the stability or shearing resistance depends also upon the temperature. That means, if you have a certain soil with a certain moisture content, this soil will show different shearing resistances according to the different temperatures. This is a very important factor. We have to consider it in our stability considerations.

I thought it might be in order to bring that out. We can have the same thing, of course in the stability of an

asphalt pavement which has been permitted to absorb a certain amount of moisture. At one temperature this moisture may not do any damage; at a higher temperature, when the fines cannot hold the moisture they have absorbed, and the water becomes liquid again, then the stability of our pavement may go down.

Consequently, the pavement may be stable at low temperatures with a certain moisture content, and at higher temperature the pavement may not be stable.

Mr. Powers: That brings up the question, as I view it, from Mr. Stoddard's point of view. He spoke of stabilized bases. Just exactly what should be the terminology for a stabilized base? Are you speaking of "stabilized" by admixtures of any nature, whether chemical or asphalt types, a cement type, or are they natural materials which may have been stabilized by the addition of either sand or clay to the natural material? I am asking Mr. Stoddard.

A. E. Stoddard: Iowa In that connection we have built a number of experimental roads which have a base stabilized with bitumen. The bitumens may be either SC's or MC's or emulsions. We have examples of each, but the major type that we have been building is a normal soil aggregate stabilization, without any treatment of calcium at all.

We have also used a base that we call a rolled stone base. It is, in effect, similar to a stabilized mixture, but made with limestone only. We have limestones which have a considerable percentage of shale in them which forms an excellent binder, and they are a very good natural base when properly laid. We have all types. The majority are of the soil aggregate mixture, clay and graded soils.

Mr. Powers: The reason I asked the question, Mr. Stoddard, is this: The designation you gave of a stabilized base brought to mind that ordinarily we don't do any stabilization of that nature. We usually put on a base course without going into a stabilized base. It is merely a stabilized layer superimposed.

It seems to me there could be a question, when you speak of a stabilized base, of bringing to mind that there are different methods that might be used by different states for the same purpose and still not be called stabilized bases. In other words, they could be called a base without the prefix "stabilized."

A. E. Stoddard: Iowa That is entirely correct. The stabilized base that we have adopted is one for our own use, principally. We have not very many supplies of natural materials which make a base satisfactory without having such a combination as would fit into that soil aggregate stabilization mixture.

Mr. Powers: Anything in which you incorporate any material of any type, regardless of what it is, is a stabilized base from your standpoint, whether it be clay, cement, sand, or any material of that nature?

A. E. Stoddard: Iowa Yes, I would say our terminology would cover any of those, but in a general way the one we refer to as a stabilized base is only the soil aggregate mixture. That is the one we generally designate as a stabilized base.

Mr. Powers: Is there any more discussion?

Prof. Housel: Michigan There are several points that have been brought up on which, if it is not imposing too much on your time, I think I would like to say a few words.

Mr. Nevitt mentioned several rational design formulae which are formulated upon two different conceptions of the distribution of the load through the base; that is, Bernard Gray's formula, which makes use of an angle of distribution or spread, and Hawthorn's formula, which makes use of an angle of spread, and then the formula which we have worked out, which makes use of punching shear.

I am just a little afraid that many of you may get the impression that here again is another one of those two opposing viewpoints which you will have to choose between and which are contradictory; that is, that they are not in agreement.

That is not the case, exactly. It is again just a little different in the selection of the controlling factors. I believe I can point that out rather briefly, and I think it might be worth while.

During the stage of load where there is no tendency to overstress the base, that is, the shearing resistance hasn't been exceeded on the boundary planes, we have a normal pressure distribution curve which is shown in all investigations of pressure distribution, in which the average angle of distribution, which I have indicated by a uniform distribution across the pressure cone is 45 degrees. The limiting angle of distribution, if you went out to the zero point of pressure distribution curve, would be approximately a 1:2 vertical to horizontal.

In such pressure investigations, as the load is increased if you are studying the pressure on any plane beneath the surface, at higher loads the character of the distribution curve changes to a high concentration in the region under the load. Various mathematical concentration factors have been used in an attempt to apply the mathematical theory of elasticity to the problem.

These concentration factors, however, vary with the depth below the load; vary with the relative load and resistance of the soil. In my work I have taken the position that there is involved here a boundary effect or a discontinuity which cannot be represented by any one mathematical equation, and that discontinuity arises from the fact that when the load becomes heavy enough to exceed the shearing resistance on those boundary planes distribution laterally no longer takes place, and the load concentrates in the central column.

Now, so far as this design of flexible surfaces is concerned, we gave up using an angle of spread because we feel the limiting value of that distribution effect is after all the punching shear. We simply say that the lateral distribution will be limited by that shearing resistance, expressed in terms of the total load distributed rather than in terms of an angle of distribution. It is simply another way of expressing the same thing.

The boundary shear factor also brings into the equation the shearing resistance of the surface, which is the essential physical property, and the thickness of the surface, which is the essential physical property, and the thickness of the surface. It brings those into the formula in a rather simplified relationship, and that is the only difference in the methods used in the Hawthorn and Gray formulae and our own.

Mr. Stoddard brought up a point about the desirability of using heavier bases. If you design a heavy base, one that is surely adequate, you are necessarily going to introduce rigidity into the base which makes the base carry the load before the subgrade develops support. If you try to establish a safety factor in your base design, then that means you are going to get away from a balanced design so far as flexibility is concerned.

There are two ways that we can go. We can either design the bases rigid enough in themselves, or thick enough in themselves to carry the total load, in which base we lose that flexibility that is desirable, or we can go to the flexible surfaces, the balanced design. Frankly, I don't know, and would not try to answer that question now, because I think it is going to require a lot more study. It seems that we should have safety factors, because the variation in subgrade bearing capacity can be so extreme; and not only that, but it can be so difficult to measure that it seems necessary that we would have to have safety factors to take care of those variations which we can't measure. That is what safety factors are for. If we could measure the subgrade bearing capacity with absolute assurance and know that it would not change, then perhaps we could use a balanced

design with more confidence. Frankly, I don't know, in that respect, which way to go; whether to follow Mr. Stoddard's lead and say that bases should be more rigid, or whether to try to keep them flexible so they will deform with the subgrade. You can't have both rigidity and flexibility and we have to make a choice in the matter of designing bases in that respect.

One other thing I wanted to cover: Dr. Winterkorn was very timely in pointing out the fact that shearing resistance depends on a number of other things than shrinkage. For that reason I believe that nothing will really take the place of a positive measure of shearing resistance, and our work has been directed in that line. We have tried to develop the shearing resistance test so that shearing resistance can be measured just as easily as other laboratory tests, and we have, I think accomplished something in that direction by the use of a very simple apparatus, a penetration machine by which we can make reliable shearing determinations right out in the field on the subgrade, on the undisturbed soil. The test is part of operations involved in taking the samples, and consists of driving the pipe to get the samples used in the laboratory, by recording the penetration of a standard core barrel under the blow of a standard weight with a standard drop. The observation takes little more time and is just as easy to do as the other methods used to get the samples for the laboratory tests, and if in getting those samples you are able to make a shearing determination, it appears to be quite practical.

Mr. Powers: Thank you, Professor Housel.

There is one other question that has been raised in connection with the summary, and that is the question of stabilizing sand mixtures. Mr. Stanton, in his paper, brought out that at least some of the fellows in the West should give some consideration to it. Since Mr. Fahnestock has written a paper along those lines, I would appreciate hearing from him.

T. V. Fahnestock: North Carolina First, I would like to say that we don't consider a pure sand mix that is mixed with rapid-curing cutback asphalt a stabilized base. In our conception a stabilized base consists of soil or aggregate of some description to which a stabilizing agent has been added for the purpose of changing certain detrimental characteristics of the material so that they will prove satisfactory under a thin wearing surface.

Rather, we think of the sand mixes as more of a bituminous mixture wherein the bituminous material serves the purpose of a binder, rather than a stabilizing agent.

I don't know whether that distinction is clear, but that is the distinction that we try to draw between the stabilized base and the sand mixes. In other words, we use in North Carolina quite a lot of hot mix sand asphalt. In that case the asphalt cement is essentially a binding agent; therefore, we consider our cold sand mixes in a similar light, although the method of construction may be different.

I don't know when the construction of sand mixes with cutback asphalt was started. It has grown up over a period of years and is quite extensively used. I believe South Carolina has used more of it than most any of the Southern States. Perhaps Florida has more now, but it has proven very successful and it is a means of utilizing an aggregate that is very easily obtainable over a wide area of the South Atlantic States, and it occurs in an area of those states where other base materials are very scarce.

The construction of the cold mix sand asphalt is accomplished now usually by traveling plants, although a considerable portion of it is still done with blades and harrows, but we think we get much closer control and much better results by the use of plants. We have successfully used within the last few years cold mix sand retreads, too, in the same areas of the state where the mixed-in-place of traveling plant mixed base has proven so successful. We are short of aggregate for use in retreads other than the sandy material, and we have used that and obtained very good results with it. It affords a cheap means of protecting the old pavements and it is very easy to construct.

The most serious difficulty in the construction of it, of course, is weather hazards. Rainy weather does slow up the construction to a great extent.

Those retreads vary from an inch to up as high as eight to twelve inches in thickness on the super-elevation of curves. A number of our older pavements did not have the curves super-elevated, and we have had very good success in super-elevating the curves with sand mix.

Unfortunately there are no laboratory tests as yet which we have found to be entirely satisfactory for designing those mixes. They are designed more or less by trial and error, and an experienced man in connection with the design of those mixes has a whole lot to do with the success of the job. Also, an experienced man in charge of the work in the field, who can tell the proper time when the application is cured sufficiently to permit it to be spread and rolled and to have the necessary stability, is very valuable.

In connection with Mr. Mills' paper, he made the statement, I believe, that from six to eight per cent moisture in sand mixes greatly facilitated the mixing. Well, I believe that is true, particularly where the sand contains a considerable portion of material passing the 200 mesh. It is also true that you can't finish one of those sand mixes with a high per cent of moisture in it unless it happens to be a pure sand subgrade. I have seen some projects built along the beach out of beach sand where the subgrade was of similar material; in such cases the mixture could be finished with five or six per cent moisture without any serious ill effects. However, on a clay subgrade it is essential that your moisture content be reduced to one per cent or lower, I think, before you can have a pavement that will resist cracking.

Of course, the closing up of the mix with excessive solvents in it has much the same effect, and it is usually a practice that, if the sand has moisture in it, the mixing can proceed with as high as six per cent moisture, and during the aeration period when solvents are being evaporated, the moisture is also lost, and I think it is very seldom that any of those mixes are closed up with more than one per cent moisture in them.

Mr. Powers: Thank you very much, Mr. Fahnestock.

Is there any further discussion? If not we will go on to the next subject that was taken up by Mr. Stanton, which was "Selection of Bituminous Materials." In that connection, as I told you in the beginning, since I prepared a paper I feel I am at least entitled to make a few comments.

Yesterday we were discussing stripping, and there are certain things that we have found out in our laboratory, although we don't believe the stripping test has particular value. However, because we have to do that particular type of testing for the Bureau of Public Roads we are interested in finding out exactly what we can about it.

In order to make a quantitative determination out of it we have found that you can take methanol and water and get a quantitative determination of stripping. I would like to hear if there is anyone else who has ever tried that particular type of work, and also what their experience is. Methanol is not a solvent for asphalt, at least not to any appreciable extent that is noticeable. For those of you who might not be familiar with methanol, it is wood alcohol.

L. C. Krchma: Missouri I have read a reference where somebody used a dye. They colormetrically measured the dyes that were assumed to be absorbed in unimolecular films on the surfaces which

were stripped of asphalt. I believe it was a German reference. They seemed quite satisfactory.

Mr. Powers: Are there any other comments on the selection of bituminous materials?

Dr. Hans Winterkorn: Missouri

I think the proposed use of the methyl alcohol is an excellent idea. We know that the stripping requirements change for location and for design of pavements. For some locations and some designs we do not need a great stripping resistance; for others we need a great stripping resistance; on the other hand, we want to make the test in the laboratory at times more severe, for instance, to get in a shorter time the result which we get in a longer time on the road, so the problem is to change the properties of water on one hand to make its action less severe, and on the other hand to make it more severe.

In respect to making it more severe, Mr. Weber tried to use alkaline solutions. Personally I have strongly warned against this; but now I think we can have, say, a system starting from methyl alcohol, have methyl alcohol mixtures with water, and then if we want to have severe conditions we can include the soaking period, as has been done by Saville and Axon. I think soon we will have a very good system for testing different degrees of adhesiveness. The methyl alcohol is very closely related to water, especially since it has the OH_3 of the water and since it is associated in a very high degree like water is. Ethyl alcohol would not be as closely related to water as methyl alcohol is, so I think this idea of Mr. Powers is an excellent one. I think one should try the experiment and see how much correlation one will have with practice.

Mr. Powers: Thank you, Dr. Winterkorn. Is there any other comment on the selection of bituminous materials?

Some of these gentlemen back here who are on the Conference Board are using different types of materials. They are using them for particular reasons. In Mr. Russell's paper he told you that he used certain materials; Mr. Finkbiner, from Oregon, uses different types of materials. If there are no other comments from the floor, I would like to hear from those two gentlemen. Mr. Russell first, please.

I. E. Russell: Wyoming

Mr. Wood has asked the question, "Is a relatively lean mat to be followed by a seal coat preferable to a richer mix." California has experimented a great deal with stability of mixes and found that when the quantity of mixing oil was kept definitely below the "lubrication point" stabilities increased. We have followed that trend

and our mixes check very closely with the surface area formula as developed in California. We are again using a large amount of SC-3 and SC-4 and are designing slightly towards the lean side. All mats are well sealed and in a good many instances stone chipped or sanded. We feel that the savings we can accomplish by the use of the less expensive SC materials together with a careful control of the quantity used, gives us almost enough to put on the sand or chip seal. We believe that we are building just as good jobs in this way as would be possible with the richer MC mats.

N. M. Finkbiner: Oregon

What we have been using on our mixed-in-place jobs is RC-3. We have been using this product for 5 or 6 years, with perfect success. In our eastern Oregon climate we prefer to use an RC-3, it is more nearly "foolproof" than the softer binders which require carefully graded materials. With an RC-3, for example, you can use materials which could not be worked successfully with an SC-2. With an RC-3 it is not necessary to control very carefully and definitely the amount of filler contained. We have had a job or two, in fact, which contained practically no filler. We seal with an emulsion or an RC-3.

I would like to return to this stripping test, if I may. It was said in one of these papers that it would be better to use an SC-4 for the stripping test. What would you think of using in the stripping test the exact kind of material that you are going to use on the road? In other words, if you use an SC-4 for the standard test in the laboratory, will that tell you the action of an SC-3 or an MC product in the field?

I would like to know what some of you think about this.

Mr. Powers:

Gentlemen, you have heard the question that Mr. Finkbiner has asked. I would appreciate it if someone would make a comment from the floor.

Dr. Hans Winterkorn: Missouri

I would think that we should test the materials that we actually use on the road. If you make a survey of the adhesiveness of your different aggregates in the state, as we are doing now, then, of course, you would choose a certain type of bitumen if you do not want to run too many tests. But on the other hand, if you do your actual tests for the purpose of establishing how your pavement will probably behave, then you should use the mixtures which you want to use on the road.

Mr. Powers:

Anyone else? Certainly there must be some of you out there who have ideas on the selection of bituminous materials. I would like to hear them.

If that is all for that particular phase of the subject we will go on to the next subject, "The Selection of the Type of Mixture."

J. H. Swanberg: Minnesota

I might say that in Minnesota our selection of mixture has depended almost entirely on the type of aggregate which we have available. In Minnesota the aggregate which is available almost throughout the entire state consists of glacial gravels. These gravels, as we find them in the gravel deposits, will contain less than 15% of –200 mesh material. In fact, most of it will run 2, 3, and 4%.

With that thought in mind we have used for our gravel road mixes and our gravel plant mixes almost exclusively, so far as construction goes, MC cutbacks and cold application tars. In our road mixes where we employ cutbacks we have used, up to this year, almost exclusively MC-2 cutbacks. But one job that we laid a year ago this fall was constructed in part with MC-2 cutback and in part with MC-3 cutback. We found on that job, (it was not seal coated) that because of the lack of fines (our void content is probably up around 15 or 20%), that after coming through the winter the section of the project, approximately ten miles long, where we used MC-2 cutback, showed a considerably higher abrasion than where we used the MC-3 cutback. This year, where the specifications were prepared subsequent to our work of last year, we have used almost entirely during the warmer months MC-3 cutback.

Our basis for selection of the cutbacks or tars instead of asphaltic oils, and the point has been made by Mr. Russell that perhaps we could have used SC materials, was that because of the lack of fines in our mixture we would probably want the harder setting materials rather than the slow-curing materials. We felt that if we were to use slow-curing oils it would involve the addition of fine materials, minus No. 200 material, in order to obtain stability in those mixtures. I was discussing that with Mr. Russell this morning, and he tells me that he studied the report on our work in Minnesota and he felt that we very likely could use slow-curing oils in those mixtures and obtain satisfactory results.

Up to the last couple of years we have done very little seal coating on these dense or open-graded aggregate mixtures; in accordance with Mr. Buchanan's definition there would be in an open-graded mixture of that type less than 5% of the material passing 200. We haven't done much seal coating on these mixes until the last year or two. We have used two types of seals. The first seal was that in which we used, usually, the same material which we used in the preparation of the mixture. If it was an MC-2 mix we used an MC-2 seal. If it was a

tar job we used a tar seal, and those first seals that we made were simply fog seals of approximately one-tenth gallon per square yard. The basis of that type of seal was this: A number of maintenance jobs were built a few years ago with that same type of aggregate where there was no scaling, and then there might be a fraction of a carload of material left and in order to dispose of that material the maintenance division applied a fog seal over the job. In some places where we were mixing the distributor might have traveled over the previously mixed surface, leaving a dribbling effect on the mixture that had already been placed. We found on some of those jobs that where we had that dribbling or that extremely light fog seal, the material remained intact with very little, if any, abrasion on it, and where we had no seal we had abrasion.

In the last year we have gone further and we are now applying seal coating material of a heavier grade and using exclusively, at least on our open-graded mixtures of gravel, a sand cover. These have proven reasonably satisfactory.

We have had one problem that might be of interest to you. We had a feeling in our office, that inasmuch as these mixtures were relatively open, we should apply our bituminous seal coat material at as low a temperature as we could and still obtain uniform distribution, and in that way decrease the tendency of the bituminous material to penetrate into our bituminous mixture.

In some cases where we had applied as much as a quarter gallon or .3 of a gallon of bituminous material as a seal coat material, in the warmer months, the seal coat material had penetrated before we could cover it. To avoid that as much as possible we kept our temperatures down as low as we could and still get uniform application. We found that even that wasn't the answer to the problem, and that applying the material at such a low temperature, although we were apparently from observation of the distributor getting uniform application, we were not obtaining uniform application, and we had a sort of streaked effect of our seal coat. About a year ago we conducted an investigation in the laboratory, using four or five different bituminous materials. We applied those bituminous materials at the minimum temperature recommended by the Asphalt Institute and at the maximum temperature of that same range. We varied the temperatures of our samples from approximately room temperature up to as high as 140 degrees, because we have found that temperature in our road surfaces. We found that we obtained as little a depth of penetration with the higher temperatures of application or even less, than

we did where we applied the bituminous material at a lower temperature.

Of course, the colder our mat was the less penetration we got. It was rather revealing so far as we were concerned, and this year we have applied our bituminous seal coating materials at as high a temperature as we can and still stay within the specification limits.

Another item that might be of interest in that same work is that within the last year we have included in our specifications for this open-graded gravel mixture requirements for abrasion of the aggregate, as measured by the Los Angeles abrasion test, and also a requirement of maximum shale content. We have limited abrasion to a maximum of 35% and we have limited the shale percentage to 5. That limit was established as a result of some work we did in the laboratory last winter, in which we varied the shale content from zero up to 10% in the gravel aggregates, adding shale to the mixtures. We mixed the mixtures with different bituminous materials and subjected the specimens to freezing and thawing tests for, I think, a week or two weeks, (I don't recall the number of cycles), freezing at 10 degrees below zero and then taking them out and immersing them in water and thawing them. We found that where our percentage of shale exceeded 5% we had considerable disruption of those samples, and we also found that where we had used cutback asphalt, MC-2, we had considerably less rupture of those samples by expansion of the shale than where we used a slow-curing oil.

Mr. Powers: Thank you, Mr. Swanberg.

There is one question which has been raised, and that is the question of what we call a seal coat. Are any of you in the audience willing to make any comments on that? Mr. Swanberg spoke a moment ago of what he called a "fog" coat. A question is also raised by Mr. Stanton and it was raised yesterday, as to whether or not sealing should be done. He expressed the opinion that sealing necessarily cut off capillary water and also cut off water that might be vaporized in the base and come into the oil surface, and as a consequence you might get destruction of the base because of the concentration of water which otherwise might be evaporated into the atmosphere.

I would like to have an expression from you gentlemen as to what you believe is a seal coat, or possibly, on the other hand, what might be termed a non-skid coat of some kind, whether it seals or not. I think we use the word rather loosely and it doesn't convey the same meaning to all of us when we speak of a seal coat.

E. B. Lockridge: Indiana

I am sure there is a difference of viewpoint as to what we mean by a seal coat. For the purpose of the paper I prepared, I said that a seal coat meant anything from a surface treatment to merely sealing the surface that had been newly completed. I think any bituminous construction requires at some stage, either immediately upon its completion or a matter of several months thereafter, a seal, and the question arises as to when does it cease to be a seal and when is it a surface treatment?

I think the question raised by Mr. Stanton as to the possibilities of a seal closing in moisture and denying its chance to escape by evaporation may be the source of trouble. We have a little evidence of that in some construction we have done in Indiana this summer under the type of stabilized gravel course on our secondary feeder road program. We finished this road, and naturally a contractor wishes to promote his work and get away from it, and it happened to be during a period when we were having considerable rain. We are having some distress showing on that road at this time. Having looked into it, we found that the sub-soil immediately below the stabilized course is saturated, and is showing distress through to the surface because of that weakened condition of the subgrade. This is an instance where probably, if we had left the seal off at least for the time being our road would have stood up better than it did with the seal.

However, in Indiana we are obliged to seal perhaps more than in some states because we have such a variation of temperature in our winters, freezing and thawing almost every day, and because of that change in temperature our surface has to be pretty well tied down or we will have raveling.

Mr. Powers: Thank you, Mr. Lockridge.

There is another question that has been raised in connection with some comments that I made on the work of Mr. Mills and Mr. Fahnestock in resurfacing old pavements, and that is on the use of an intermediate layer of material between an old pavement and any resurfacing that you may want to do. Down in our state it worked very successfully. The maximum size rock we use is half inch, and it is half inch to dust. The usual type of material we use is crusher run passing that particular size, and ordinarily would comply with all the limitations that we have for the ordinary dirt road so far as surfacing is concerned—liquid limit probably in the neighborhood of about 25, plastic index probably not to exceed 3, and uniformly graded.

I would like to know if there is anyone who has

had any experience with that particular type of construction and whether they consider it to be satisfactory or not.

There is another question, since I can't get an answer to that one: Mr. McKesson, in his paper, has this to say: "It is here asserted that the penetration test alone is the least important of all physical tests of asphalt and that narrow specification limits in penetration on any grade of asphalt are purposeless. With wide allowable specification limits, such as 50-100, 100-200, 200-300, and 300-350 penetration, the test is valuable for classifying asphalts into various grades by relative consistency."

He also goes on to say that he doesn't believe the present ductility test is satisfactory for determining the suitability of asphalt, nor is it suitable to determine the eventual ductility, and says that some type of ductility tests should be made on mixtures rather than on the asphalt itself.

I would like to have Mr. Wood comment on that, please.

W. H. Wood: Texas

I have never felt that ductility was a necessary quality for asphalt. I know that I will probably strike fire somewhere there, but I have seen penetration roads built out of roofing asphalt which had a ductility of around 4. I don't know what the penetration was, but the aggregate on that job sticks very satisfactorily.

Some time ago I made a survey of the use of liquid asphaltic products in Texas, consisting of a survey of most of our mileage, particularly in western Texas. We found in this survey that although the ductility may have been 100 or 125, or it may have been 50 when the asphalt was originally put down, samples taken from the jobs, all of which were at least two years old, brought into the laboratory and extracted and the asphalt recovered therefrom, showed ductilities varying from around 25 or 30 on up to 110 plus, and that there was no direct relation that we could establish between the ductility of the recovered asphalt and the behavior of the job in service. Those jobs were classified as "A," "B," "C," and "D", depending on whether they were satisfactory or not.

We have always run ductility tests. We have argued with lots of people and with ourselves as to whether they have any connection with the adhesiveness of the asphalt. Personally, I don't think that that is the case. We know that the asphalt does lose penetration and in some cases it does lose ductility in service.

We also ran, in that series, reclaimed samples on each of those cars of asphalt represented by each of those

samples. We found that in storage in half-gallon cans the characteristics had changed very little during a period of as much as three and one-half years, which is the maximum time we kept the samples.

I want to say again that I don't think ductility is necessarily an indication of quality. We specify one, but simply because we don't know what else to specify. I think you are all familiar with the fact that we have no quality specifications whatever for asphalt anyhow. We merely describe the product that we have been using in the past and feel that if we get material that has the same general characteristics we probably will get the same general behavior. One probably will. It is going to depend on how you use it as to what kind of results you get, anyhow. I think the biggest challenge we have is to know what asphalt is supposed to do, what it actually does in the job.

We in that survey found that there was no relation at all between the classification given the jobs and the time of the year in which they were put down, yet we are firm believers in putting asphalt down during "cotton" weather. We think if it is hot enough to be good for a cotton crop it ought to be good for putting down asphalt also. However, a lot of jobs put down in the winter time came out with as good a classification as those put down in the summer time.

In addition to that you find material being shipped simultaneously from the same plant and going to different jobs will result in classifications, some of which are quite satisfactory and others of which are unsatisfactory. As I told one plant chemist, I didn't think they were smart enough to separate the cars. I thought the asphalt going out to the various jobs was all of the same quality, whether good or bad; yet the resulting quality was of wide variation, all under the same specification so far as construction was concerned.

That is not an indictment of engineering; it is some factor in the construction of an inverted penetration job which was being largely overlooked in construction, and that is, of course, whether or not it is going to invert. We don't know. If it does, it will make a good job and not have to be sealed. If it does not, it will probably ravel on top and it will have to be sealed eventually.

In connection with sealing, I was very much interested in what Mr. Stanton had to say about the fact that open-graded mixes and even dense-graded mixes would pass water vapor from the base in sufficient amounts to affect the quality of the base. We have a large mileage in our state of inverted penetration asphalt.

and a few other hot mixed types, all of which we feel are highly impervious to moisture. In most of our penetration jobs, particularly those which did not invert sufficiently to keep the moisture from the seal coat, we found we had moisture in the bottom of the mat from one-eighth to a quarter of an inch thick.

I don't think it is possible for water vapor to go through the asphalt mat. If it is possible, however, to build a cold mix mat which will permit the passage of some water vapor, enough to have an appreciable effect on the base course material, I personally will be very much interested in finding out that fact, because we do have borderline materials that we use for base courses and we realize that without a surface course that base course would behave satisfactorily so far as stability is concerned, but probably upon being covered by an impervious surface we quite often accumulate enough moisture to render the base instable.

If it is possible to design a mat which will let enough water vapor out to keep it from concentrating so that the base loses stability, I am personally going to look into it. I am interested in it.

Mr. Powers: Thank you, Mr. Wood.

Are there any other comments? We still have a few moments more, so far as the time allotted to us is concerned, and if there is any other subject, we can discuss it. We have pretty well covered the subject so far as I am concerned as chairman of this section, and unless there are other comments that you fellows would like to make in connection with the papers yesterday or the papers today.

W. H. Wood: Texas I want to ask a question, and if you want to leave, just don't answer it and we will be through right quick.

We have in the State of Texas a large mileage of roads, concrete and of asphalt for instance, that were built on bases that are probably in most cases about a foot wider than the surface. We are confronted with a proposition of shouldering those jobs economically. We, of course, can go in there and dig out a trench or dig the whole top of the road off, or build another job beside it and call it a shoulder. That is a pretty expensive prosposition. We want to find some way of stabilizing the soil material which lies adjacent to the edge of the existing base, whether it be the concrete slab itself or the base of the asphalt road.

There are two reasons for that. One is to provide some transition from the present surface to the outer

shoulder, and the other is to provide an impervious blanket which will tend to keep the moisture content more constant under the existing pavement, both of which are desirable characteristics.

We hear a good deal about stabilization out of the state. All that we have been able to find out about so far has been about stabilization of sand or gravel materials. If any of you have had experience in stabilizing clay with any kind of asphalt materials I would like to hear you discuss it. I see some grins around the audience. I am serious. We have lots of that stuff and we really want to do something with it, without just hauling it off the road and having to replace it, and I think the rest of you probably have quite a lot, too. If any of you have had experience, positive or negative, on a design of mix for use with A-6 or A-7 materials, we would like to get an expression from you.

Mr. Powers: Thank you again, Mr. Wood. You gentlemen have heard the question he has propounded to you. I personally would like to have some answers myself.

Mr. MeLeod, can you give us any information?

N. W. McLeod: Saskatchewan That is a question I would like to hear answered myself. Certain chemical treatments have been proposed, but these have not yet received the practical field tests required to prove their utility.

Mr. Powers: Shreve Clark, have you an answer?

Shreve Clark: Virginia Our experience has not covered a sufficient period of time. We have about six miles of roadway consisting of A-7, A-4, and A-2 soils, two miles, consisting of A-7 and A-4 stabilized with about 9% of Portland cement. We obtained between eight and nine hundred pounds per square inch on cylinders from this work. This section was stabilized with 13% and 17% moisture and, of course, is showing some cracking, but we are putting on a surface treatment and only time will tell how long it will stand up under traffic.

We also stabilized a two mile section with a tar, RT-4, using 14% of moisture and 5 to 6% of tar. That section is cracking rather badly and is not very satisfactory. We stabilized a third section about two miles long, using 3% of moisture and 6 to 7% of MC-2, which is showing up much better. This mix was designed for maximum stability rather than minimum density; therefore, the results seem much better due to the low shrinkage. All sections will receive a bituminous treatment. Only time will tell if these types of stabilization of A-7

soil will prove successful. We have many miles of this material and will have plenty of experience with it in the next few years.

Mr. Powers: May I ask you a question before you are seated? How thick did you treat?

Shreve Clark: Virginia. Six inches.

Mr. Powers: In each case?

Shreve Clark: Virginia In each case. There are both cuts and fills on the A-7 soil sections. It is fairly uniform. This work is in a county where practically the entire county is of that type of soil, with a high silt content. The clay content is not so very high, but the silt content is. The maximum size of the sand is 20 mesh, and the amount passing the 200 mesh is 40 to 50%. When it gets wet there is no bottom, and it also is very slippery. Therefore, we are trying out various forms of stabilzation to see which is best for that particular road.

Mr. Powers: May I ask another question? Do you know anything about the characteristics of the MC you used, whether is was positive or negative?

Shreve Clark: Virginia. No.

Mr. Powers: Before you continue any further, Mr. Wood wants to ask you a question.

W. H. Wood: Texas Did you say your MC section has been surfaced or is to be surfaced?

Shreve Clark: Virginia It is to be surfaced. It is probably surfaced by this time.

W. H. Wood: Texas You will probably find out very quickly whether it is any good.

Shreve Clark: Virginia We will probably find out a lot of things in the next couple of years.

Mr. Powers: The type of soil Mr. Wood and I have in mind would run probably 75 to 90% minus 200, instead of 30%.

Are there any other questons?

I. E. Russell: Wyoming Mr. Stoddard discussed stabilized bases, and I would like to know what method they use for determining the amount of soil binders to use so as to predetermine what the strength will be in the base.

A. E. Stoddard: Iowa The amount of soil is determined by the grading of the material and the plasticity index you are trying to get from the material passing the 40-mesh sieve in the final

mixture. When we first started building those roads we were using a P. I. of around 8 to 15, I think was the range of the specification. The Bureau now asks us to work between zero and 6, and we think that somewhere between about 4 and 10 would be about all right. We have a few roads with a fairly high plasticity index which were laid in 1934 and still seem to be giving perfectly satisfactory service. There is an experimental road in Cass County about ten miles long in which we have a soil that I guess can be classed as A-7, that has about 80% passing the 200-mesh, on which we have constructed experimental sections. One of them is treated with tar, another with MC, another with SC. There is also a section of cement stabilization on that same road, and there are sections of base with soil aggregate mixtures. Those have all given fairly stable bases. There is also a section with an emulsion as a stabilizing agent. The sections were surfaced this year and, as with Mr. Clark, in time we will know better what happened to them.

In connection with the emulsion, it was laid very late in the fall and retained practically all its moisture until early in the spring, at which time it looked very good at first, but it began to develop some cracks in the surface as the stuff started to dry out. That was before the final surface was put on. The last time I saw it, it showed a little bit of tendency to be a little soft because apparently it had not got rid of enough moisture, but other than that the rest of the surfaces all look good; we can tell better later.

Mr. Powers: Are there any further comments in connection with stabilization or any of the subjects that we have covered? Are there any questinos?

J. H. Swanberg: Minnesota

We have had considerable discussion in Minnesota, particularly with the Bureau of Public Roads, in which the question has come up as to what effect the use of calcium chloride admixture in a clay stabilized gravel would have on the penetration of the bituminous material. Based on our own experience, from observations in the field, apparently there isn't much of any effect of the calcium chloride on the penetration of bituminous material.

I would appreciate very much receiving any thoughts that anyone might have as to what effect, if any, the calcium chloride admixture will have on the penetration and possibly the durability of that bituminous material.

H. G. Nevitt: Missouri

Do you not penetrate a year later? Is it done the same year?

J. H. Swanberg: Minnesota

Yes.

T. R. Perry: Iowa

We have two sections of stabilized base, one with calcium chloride used as an admixture and the other without. They are contiguous to each other and made from the same aggregates, except the calcium chloride. Tar prime was used and the penetration was good. The prime was put on late last fall and there was no peeling. The road went through the winter on the prime only. We had rather severe peeling this spring on the calcium chloride section and it was necessary to do considerable repairing before the surface treatment was put on.

Mr. Powers: Dr. Winterkorn, could you add anything to that, please?

Dr. Hans Winterkorn: Missouri

Personally, I could not say much about the penetration, but I could add a few remarks on the change of the affinity of the soil for bitumen by treatment with different salt solutions like calcium chloride, sodium chloride, barium chloride, magnesium chloride, and so on. We used eight different chlorides and then washed out the excess of the salts so that in the main we had ionic exchange.

But on the other hand, there always will be a certain amount of salt or calcium chloride left in there which probably resembles the condition in that road, say a week or two after the calcium chloride has been applied.

We have found that, in general, calcium does not change very much the affinity of the soil for bitumen, either in a good or in the bad direction. With some northern soils it increases the affinity; with other northern soils it decreases it slightly. With other salts, as with soduim chloride, we found a definite decrease of the affinity. With salts like aluminum chloride and iron chloride we found a definite increase of affinity.

This whole investigation, which comprised six soils from various regions of the United States and two soils from the Argentine, eight ionic treatments, and eighteen different asphaltic materials and five different tars, was quite involved and we spent about four years on it. It will be available by the end of the year, so if you want to study it, it might be worth while to do so, because of the possible effect of calcium chloride or sodium chloride treated bases or other treatments. I think you might look forward to it by the beginning of next year, when it will be available.

N. W. McLeod: Saskatchewan

From actual experience, the presence of calcium chloride, sodium chloride, or sodium sulphate in stabilized gravel mixtures in Saskatchewan has had no apparent influence upon the penetration of primes into the surfaces into which these admixtures were incorporated. During the months of May, June, and July, the average relative hu-

midity in Saskatchewan is quite low, and calcium chloride applied even at the rate of two tons per mile per week has been entirely ineffective for maintaining the road surface in a damp state.

Mr. Powers: Are there any other comments? It is within a few moments of the time we are supposed to adjourn so, gentlemen, if that is all, we now stand adjourned.

Papers Prepared

on

Fundamentals of Bituminous Construction

for

The Montana National Bituminous Conference

The Place of Seal Coats in the Fundamentals of Bituminous Surfaces

By

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In undertaking to write this paper, I am not presuming that the subject of Seal Coats would be considered first, of the fundamentals of Bituminous Construction. However I believe that it is one of the most neglected and overlooked. Particularly do I think this is the case in County work, but it also applies in less measure, in all construction work of various types of bituminous surfaces, in the Central and Middle Western States. It seems to be like the weather, much talked about, but little done about it

I do not presume that I will say anything new as all of these points have previously been discussed by able writers, so it is likely that I will only restate what has previously been brought out, with the wish that this very important fundamental will receive more attention.

A very common Seal Coat in this section is found where the surface is constructed by using stone of ¾ inch maximum size and apply about 10 pounds per square yard of ⅜ inch maximum stone chips and compress into the voids made by the larger stones, without any additional bituminous material being added to the smaller chips or surface. Often times in using aggregate of ½ inch maximum, the road is simply rolled with no further sealing attempted, in either case leaving an entirely too open a surface.

The particular Seal Coat I would like to speak of is with road mix type, using aggregate of the following size:

	Aggregate For Seal Per Cent by Weight
Passing ½ inch sieve	100%
Passing ⅜ inch sieve	90-100%
Passing No. 10 sieve	0-5%
Passing No. 200 sieve	0-2%

or preferably size:

Passing ⅜ inch sieve	100%
Retained on No. 3 sieve, not more than	15%
Retained on No. 10 sieve	20-45%
Passing No. 40 sieve	7½-17½%
Cutback Asphalt	3-5%

and rolling to a very compact mat, and after traffic has kneaded this surface for at least 10 days, make the following application:

First be sure that all foreign and loose material is swept from th surface, then apply not to exceed 15/100 of a gallon of Bituminous Materia (preferably one of the emulsified asphalts) per square yard and if the fine grade of stone is used, 1/10 gallon will be ample, and cover immediately with sand to the amount of 10-15 pounds per square yard or more i the voids will hold it. In sections where refuse sand from gravel plants i available this refuse sand will serve the purpose making this additiona sealed coat very inexpensive, yet it accomplishes the following purposes

(a) First it makes a surface as near water proof as possible. While I realize water in itself is not so detrimental to bituminous surfaces, yet in middle and northern states often times these surfaces are saturated with water in freezing weather. This freezing of water within a porous mat is very destructive causing the pushing apart of the aggregate and causing it to peel and ravel. This sealing against water has often times been considered very important to the extent that excess crowns have been used to assist in water proofing the surface. However it is one of the least destructive elements in itself unless in sections where sleets and sudden freezing would occur.

(b) It gives a sandpaper finish which is according to Professor R. A. Moyer of Iowa State College in his report on "Skidding Characteristics" the nearest skid-proof surface that can be constructed. This feature alone has been too often overlooked. Many have held to the false idea that the rough surfaces were skid-proof but by the very extensive research work made by Professor Moyer, it has been definitely proven that of all types of bituminous surfaces the sandpaper finish is the most desirable and the nearest non-skid surface to construct. It also gives a smooth easy riding surface eliminating the roar in cars that so often is the case with a rough surface.

(c) Third and by far the most important reason for a properly sealed coat is to make them as impermeable to air as possible and thus prevent circulation of air within the mixture. The worst enemy and the most destructive factor of the natural forces to bituminous surfaces is air. This sealing against the circulation of air within the mat has been very much neglected, in many cases through a lack of information. In fact in the construction of many surfaces where all other fundamentals have been closely followed, then to leave an open porous top is like building a house forgetting the roof. Air oxidizes the film of asphalt surrounding the particles of aggregate causing it to dry out, harden, and become brittle and lifeless. By the elimination of air, little or no hardening should take place after construction or at least the hardening process is eliminated to the greatest possible degree. This is true in all types of bituminous surfaces and offers an explanation for the hard brittle condition of many open texture, low cost bituminous surfaces after a short period of service and this can in many cases be prevented by proper Seal Coat.

In conclusion I will say that too much emphasis can not be placed on the elimination of air within our bituminous mats, add to this the non-skid surface for the small additional cost of 1/10th of a gallon of bituminous material with a liberal application of cheapest sand and I am convinced it is a good investment.

Interdependence of Sub-Base, Base and Surface in the Design of Bituminous Pavements

By

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A paper on this subject almost inevitably must begin with the trite statement that all traffic loads are carried by the subgrade. The statement is certainly overworked, but its significance is too often overlooked. Whatever, in the nature of base and surface, is laid serves primarily to distribute the load over the subgrade. If the ability of the superstructure to distribute the load is adequate all will be well; if inadequate, failure will result.

Thus, whether or not it is consciously recognized in designing a road, the load carrying capacity of the subgrade is always taken into account, for no one deliberately designs a road for failure. It is true that cases are often cited of roads of the lightest and cheapest construction which successfully carry traffic. The stories of their serviceability have been told in awestruck tones as being outside the realm of explanation. Instances of such surprising behavior do exist, but should not be regarded as supernatural phenomena. Road builders deal with forces and materials which behave in accordance with the principles of mechanics. According to the laws of statics the condition of equilibrium is attained only when applied loads are resisted by other forces equal in magnitude and opposite in direction. If a road does not fail under the loads of traffic equilibrium exists. The apparent sweet mystery of success is due to the normal functioning of factors and conditions which are not recognized or understood.

The natural soil subgrade is common to all roads, but all soils are not the same. All possess bearing capacity to some degree, depending upon composition, texture, etc., and also upon the amount of water which they contain. Clays and silts, the most troublesome soils, decline rapidly in supporting power with increases in moisture content up to the point of saturation. Thus it seems that the soil constituting the subgrade should receive the careful attention of the designer before bases and surfaces are considered.

This line of action is being followed in many states and the soil survey is destined to become as much a matter of routine as the line survey. Michigan is one of the pioneers in this movement and has adapted the U. S. Bureau of Chemistry and Soils agricultural classification of soil types to highway uses. Plans designate the nature and extent of the various soil types along the proposed route. Each type has its typical profile with varying textures, drainage characteristics, height of water table, etc. With this information at hand, designing engineers are able

to prescribe drainage where it will be effective and to establish the location and grade so that most favorable soil conditions will prevail.

Although this is a long step in the right direction the information obtained from the soil survey is qualitative rather than quantitative. Experience still is the guide in determining the extent to which the subgrade may be depended upon. Subgrade bearing capacity has been roughly estimated in the past for design purposes, but until lately few serious attempts have been made actually to measure it and determine the additional resistance required to carry traffic loads.

Published articles on flexible road design by Hawthorn of the University of Washington and Housel of the University of Michigan have attracted a great deal of attention. The former experimentally derives a shearing angle for the surfacing layer which defines a circle on the subgrade over which the load is assumed to be distributed uniformly. This angle varies, as consequently does the circle on the subgrade, with the material, the thickness of surfacing layer, and the nature of the subgrade. Hawthorn seems to have worked with intensities of load smaller than those which cause failure, for it is well known that at higher magnitudes, approaching the limit of resistance of the road structure, there is a concentration of stress in the region directly under the area of application.

This region is called the central column by Housel, who pictures failure as occurring when the combined shearing resistance of the surface layer and the subgrade around the surface of the central column is exceeded. At this moment the ability of the materials to extend the distribution over an area greater than the loaded area ceases, and failure occurs by punching shear. The validity of this analysis of the mechanics of flexible roads at the time of failure apparently has been substantiated by field tests by the Michigan State Highway Department. Plant mixed surfaces of the graded aggregate type constructed over existing gravel roads on both clay and sand subgrades were tested to failure by means of loaded bearing plates. Likewise on adjacent areas the uncovered gravel base and the subgrades were tested.

It seems quite certain that by this method the thickness of courses above the subgrade necessary to support known loads can be rather definitely calculated by the application of a simple formula based on the properties of the materials in question. The strengths in shear of the subgrade and of the superimposed courses must be known. Obviously the subgrade should be investigated in its least resistant condition which, for plastic soils, implies the highest moisture content which it is likely to contain.

The selection of the number and type of courses to be constructed to supplement the strength of the subgrade depends upon considerations of economics. Stabilized soil, gravel, crushed stone or bituminous mixtures may be used individually or in combination. Naturally the surface is likely to be bituminous as insurance against water entrance as well as to resist traffic wear. Little information regarding the shearing resistances of surface and base mixtures can be given at present, although it is not particularly difficult to develop in connection with any proposed type of construction. The Michigan field tests indicated shearing strength of between 35 and 45 pounds per square inch for a superimposed layer made up of 5 inches of compacted gravel and 3 inches of graded aggregate type

urface. The shear strength of the clay subgrade was by test determined o be 4.5 pounds per square inch but appeared to be considerably in- reased, perhaps to 6 or 7 pounds per square inch, due to the resistance o displacement of the gravel and bituminous courses above it.

It is important to note that the required thickness of base or surface ourses is dependent upon the shear strength of the mixture of which they re composed. With any mixture or aggregation of particles shear strength ncreases with the intimacy of contact or interlocking of adjacent particles. Therefore, careful gradation in design and maximum compaction in con- truction are decidedly advantageous.

Roads of the flexible types have been built and have carried traffic or many years without more than a superficial knowledge of subgrade onditions or of the structural characteristics of the base and surface ourses. Many others doubtless will be so built in the future. It is natural o ask whether it is necessary to attempt as careful design as seems pos- ible in the near future, or even now. Is it practical to vary the thickness of base or surface from place to place in accordance with variations in the upporting value of the subgrade? Is the adoption of a method of design ikely to result in marked economy?

It is hazardous to answer such questions categorically. It must be recognized that the trial and error method has worked more or less suc- essfully with flexible types which can be thickened or subjected to ex- tensive repair over those areas which service demonstrates are inadequate. It probably would be uneconomical to vary the thickness every few hundred feet, but careful subgrade investigation should indicate the existence of subgrade which is below the average in supporting capacity and afford the opportunity to correct, by drainage or otherwise, such areas before base and surface construction begins.

Whether or not savings in first cost can be made will depend upon present design standards and local subgrade conditions. Observation leads the writer to feel that over clay subgrades present practice in many localities is to construct flexible type roads which are dangerously close to the minimum requirements for bearing capacity. In such cases this situation would be clearly indicated by the evaluation of subgrade bearing capacity which would probably lead, in future design, to the application of a reasonable factor of safety in base and surface requirements. This would probably increase construction costs, but maintenance should be ma- terially reduced and the road should be more satisfactory from a service standpoint.

On the other hand, there is evidence such as the occasional spectacular performance of extremely thin surfaces that general practice may tend toward over-design in the case of superior subgrades confined by base and surface courses. Such subgrades have been found to possess a bearing capacity several times that of clay subgrades. Assuming the same types of surface and base courses and the same traffic loads, it is obvious that less thickness is required and savings can safely be made.

There are possibilities of economic advantage in the application of definite knowledge of the strength characteristics of subgrade and flexible type mixtures, but these are incidental to the rational treatment of any specific set of conditions which, as in other fields, may be called balanced design.

Premix Seal Coats and Light Surface Coating of the Graded Aggregate Type

By

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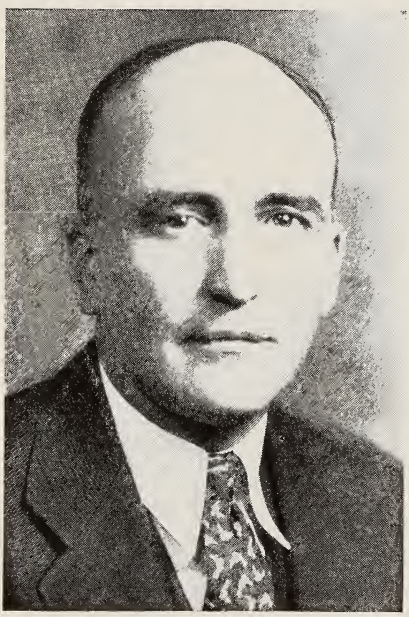


The subject of premix seal coats is one of particular significance to highway engineers at this time as a large mileage of rigid type pavement constructed during the twenties has reached the point where some resurfacing is essential to provide protection against the ravages of traffic and weather.

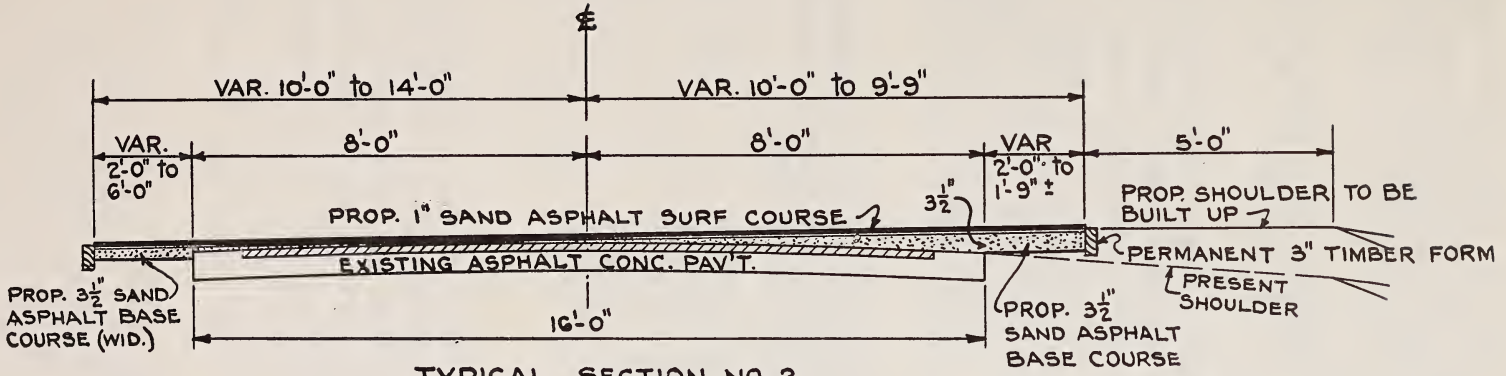
In addition to this large field there is no doubt that premixed material has certain advantages for use on low cost roads both as a seal on new work and for resurfacing.

North Carolina with 4,500 miles of surface treatment on soil type or macadam bases, 1,200 miles of low cost bituminous mix pavement which is hot and cold sand asphalt, and 900 miles of bituminous concrete, has a maintenance problem of considerable magnitude. It is this large mileage of bituminous pavement which makes it essential that the resurfacing work done be designed to protect the investment at the lowest possible cost.

It is recognized that premixed plant materials are usually preferable to road mix methods for resurfacing but the cost of the plant mix material, which is usually at least twice as much per ton as road mix, has precluded the use of plant mix for resurfacing to any large extent in North Carolina.

The writer believes that with more efficient mixing plants and improved spreading devices the price margin can be greatly reduced and that premixed material will be used more than in the past.

Among the plant mix types which have been successfully used is Downard Rock Asphalt or Crown Rock, J Type. The methods used in preparing this mix consist of blending a mixture of aggregate and oxidized asphalt, which has been chilled after mixing, with a mixture of aggregate and asphalt flux, oil asphalt or fluxed Bermudez asphalt. The mixture is completed by combining the two mixes in approximately equal proportions in a separate mixer or blender. This material can be readily spread



TYPICAL SECTION NO. 2

SECTION SHOWING METHOD OF SUPERELEVATING CURVES & WIDENING WITH SAND ASPHALT BASE COURSE

NOTE.—

WHEREVER THE DEPTH OF THE PROPOSED SAND ASPHALT BASE COURSE IS TO BE PLACED EXCEEDING 3½" IT SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE SPECIAL PROVISION FOR "SAND ASPHALT BASE COURSE WITH COARSE AGGREGATE

after storing for an indefinite period of time and is workable even in cool weather. Very satisfactory results have been obtained by laying as little as 50 pounds of this material per square yard, both for resurfacing old roads and as a seal on new surface treatment.

One of the most economical plant mix types which is used in North Carolina is hot mix sand asphalt. This material is now being used to widen and resurface old sand asphalt and asphaltic pavements, twenty miles of this work having been completed during the early part of this year.

The attached typical section illustrates the method of widening and superelevating curves. The construction procedure on this work is as follows:

The trench for widening is excavated and wooden forms set in place. The subgrade for the widening is thoroughly rolled with a trench roller, the sand asphalt base course material is then placed and rolled; when this has been completed for the entire length of the project, a squeegee coat is applied to the surface of the old pavement and the widened area, following which a sand asphalt leveling course having a minimum thickness of ½ inch is placed and a second squeegee coat applied before the surface course is placed. The surface course has a thickness of one inch.

The grading of the sand for this type of work is varied within wide ranges so as to take advantage of local sand pits as much as possible. A typical composition for the surface course is shown below:

Passing	Retained On	Per Cent of Total
No. 4 Sieve	No. 10 Sieve	0 to 8
No. 10 Sieve	No. 40 Sieve	5 to 50
No. 40 Sieve	No. 80 Sieve	20 to 75
No. 80 Sieve	No. 200 Sieve	5 to 35
No. 200 Sieve		4 to 8
Bitumen		7 to 10

The bitumen used is ahphalt cement having a penetration at 77° F. of 85 to 100. For the base and leveling courses the bitumen content is reduced to from 5 to 7 per cent and the 200 mesh material, to from 0 to 5 per cent. Where the leveling course is to be thicker than 3½ inches due to superelevation of curves, at least 50 per cent of coarse aggregate passing the 1 inch and retained on the No. 10 is used in the mix.

The above limits are of course master ranges and are subject to closer control by a job tolerance.

The use of 85 to 100 penetration asphalt together with the small percentage of 200 mesh material produces a pavement which is likely to mark slightly under horse drawn traffic during the first summer but these marks are soon obliterated by pneumatic tires and it is believed that the ultimate life of the pavement will be greatly increased over mixes using lower penetration asphalt and high filler content. So far it has not been found necessary to seal the hot mix sand asphalt until

it has been in service for five years or longer as the texture of the pavement surface, while tight enough to resist weathering remarkably well, is practically skid-proof.

The trend toward the increased use of premixed material for retreatment work is illustrated by the Virginia Department of Highways. This Department awarded contracts for approximately 165,000 tons of plant mix material in February of this year. This material is used for resurfacing macadam and concrete pavement and excellent results are being obtained by using about 120 pounds per square yard. Most of the material used is of the liquefier type although all of the types in current use are placed in open competition.

In South Carolina the use of premixed material for seal and retreatment work has been very limited. This is probably due to the fact that the South Carolina Highway Department has obtained excellent results through the use of road mix methods.

In the maintenance of bituminous surface treatment it is necessary to apply retreatments at intervals of from three to five years. To adhere to this schedule, North Carolina must retreat 1150 miles each year. This retreatment work is performed with State owned equipment and by trained crews in each of the ten divisions.

The type of retreatment applied on different roads varies of course with the condition of the road and the availability of local aggregate but can be classified under two general headings, drag retreatments and mixed in place retreads. Rapid curing cut back asphalt is used exclusively for this work in North Carolina and the specifications for this material contain requirements for testing the base asphalt and naphtha solvent as well as the finished product. A close control of the viscosity of the material ordered is also required. Current practice is to use material having a Furol viscosity at 122° F. of from 120 to 160 for work done in the spring and fall, while for work done in mid-summer the viscosity is from 200 to 250.

The drag retreatment consists of from 25 to 40 pounds of aggregate, graded from the half inch sieve to the No. 16, mixed in place with from .35 to .5 of a gallon of rapid curing cutback asphalt per square yard. The quantity of aggregate and asphalt used is varied according to the condition of the surface being treated. The mixing is accomplished with a long base broom drag 12 feet wide. The drag is equipped with 5 rows of wire brooms and is hinged to permit it to conform to the cross section of the road. These drags are mounted on wheels to facilitate turning and moving from job to job.

The treatment is constructed by applying a tack coat of cut back asphalt to the clean surface of the pavement, spreading the aggregate and allowing it to dry, then applying the required amount of cut back asphalt, mixing and rolling. When it is necessary to open the road to traffic the same day, 5 to 7 pounds of coarse sand or chips are spread over the surface and rolled in.

One outfit consisting of distributor, broom drag, trucks, roller, etc.,

can place from twenty to twenty-five thousand square yards of this treatment per day.

The drag retreatment can be used over all types of pavement and gives a uniform non-skid surface at a cost of from eight and one-half to eleven cents per square yard.

Mixed in place retreads are used where it is not only desired to smooth up the old surface but also to provide a certain amount of added slab strength.

The aggregate used in retread work consists of sand, or a mixture of sand and stone, screenings and stone or sand gravel.

Where local sand of reasonably good quality and grading is available, sand retreads are used. The grading of the sand on the #10, #40 and #80 sieves does not appear to be of vital importance, if the sand is not all of one size, as satisfactory results have been obtained with sand which varied widely in grading on these sieves. However, it is important that the percentage passing the #200 sieve should be less than ten per cent, as over this amount of fine material causes difficulty in mixing. Also, it is important that the sand grains be angular in shape and not rounded as retreads using sand having smooth regular surfaces on the grains will not have sufficient stability to withstand heavy traffic or parking. A microscopic examination of samples of sand proposed for use is usually made to determine the shape of the grains.

Retreads are placed from one-half inch to three inches in thickness and the construction procedure is to spread sufficient aggregate on the pavement which is to be given the retread to provide the desired compacted thickness. The aggregate is then dried, if necessary, by manipulation with blade graders until the moisture content is 4 per cent or less. Rapid curing cut back asphalt is then applied with a distributor and mixed with the aggregate with blade graders or retread machines until the mixture is uniform in color and consistency and until the evaporation of the solvent and any moisture in the aggregate has progressed to such a point that the mixture will be firm and unyielding when spread and rolled. Unfortunately there is no accurate field test for determining when the mixture is at exactly the right stage of curing to allow spreading with the best results. This is a matter of experience and can be determined best by the "feel" of the material.

Just prior to spreading, the mix is placed on one half of the surface to be covered and a tack coat of cut back asphalt is applied at the rate of .15 of a gallon per square yard. The mix is then bladed on this portion of the road while the tack coat is applied to the other half of the width to be resurfaced. As soon as the mix has been smoothly spread with blades, it is rolled. Traffic is not permitted on the work until the mix has cured so that displacement will not result.

The bitumen content used in retreads is from 4 to 6 per cent, depending on the grading and the cost is from $2 to $5 per ton.

Where it is decided to use a mixed in place retread the question naturally arises as to the thickness which should be used. This is de-

cided by a study of the pavement to be resurfaced. The thickness generally used on old surface treatment is variable as some of these roads were constructed with much more crown than is now safe, therefore, the retread is often placed three or four inches thick on the edges with only one-half inch or less in the center. On asphaltic concrete and cement concrete the thickness used is usually one inch while on old hot mix sand asphalt, two to three inches is used. The reason for the thicker retread on the sand asphalt is that on these pavements built ten years or more ago and on which low penetration asphalt was used, contraction cracks are numerous, some of these being ¾ of an inch wide.

It has been found that these wide cracks will show through a one inch retread in about six months whereas two or three inches of retread will seal these cracks for several years.

Study of Strength of Asphaltic Concrete Mortars

(Progress Report)

By

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In the past few years considerable interest in the disintegration of asphaltic pavement has been evidenced by a number of research projects on this subject. In these investigations, it seems that more emphasis has been placed upon the nature of the cementing medium than upon grading of the mineral aggregate. The purpose of this investigation will be to endeavor to determine the proper relationship between the coarse aggregate, the fine aggregate, and the asphaltic cement of the densely graded type. Work being done is a study of mixtures of fine aggregate and asphaltic cement, particularly the relationship of mineral filler and asphaltic cement.

After a review of the many methods used to predict the service behavior of asphaltic paving mixtures, the compression test *(1, 2, 3) was selected as being the most suitable. For the present work, the fine aggregate specifications of the Oregon State Highway Department were used as a basis for the fine aggregate gradings as follows:

Passing No. 10 sieve and retained on No. 40 sieve	30% to 50%
Passing No. 40 sieve and retained on No. 80 sieve	16% to 32%
Passing No. 80 sieve and retained on No. 200 sieve	16% to 32%
Passing No. 200 sieve	8% to 16%

This grading was divided into two parts—sand and mineral filler. Hereafter, that portion passing the No. 10 sieve and retained on the No. 200 sieve will be termed sand, and that portion passing the No. 200 sieve will be referred to as filler.

The material used was manufactured from available quarter inch minus aggregate. This was pulverized in a disc-type grinder and separated into the aforementioned sieve sizes. The sand for each 5000-gram batch of mixture was then recombined in the following proportions:

*Bibliography

Passing No. 10 sieve and retained on the No. 40 sieve 45.0%
Passing No. 40 sieve and retained on the No. 80 sieve 27.5%
Passing No. 80 sieve and retained on the No. 200 sieve 27.5%

This mixture was combined with the desired amount of mineral filler and heated on a hot plate to 400-450° F. The aggregate mixture was then placed in a 500-gram capacity covered pug mill and mixed until the temperature dropped to 350° F., whereupon the desired amount of 51-54 penetration asphalt, heated to 250-300° F., was thoroughly incorporated and stored in a closed container and held in an oven which was maintained between 250 and 300° F., until ready for use.

Five compression test specimens, 3.4 inches high by 4.5 inches in diameter, were made of each separate mixture, as follows:

The hot mixture was placed in a steel mold and compressed under a load of 5000 pounds per square inch. This load was maintained for ten minutes during which time water at about 60° F. was circulated around it. The specimen was removed from the mold, placed on a smooth steel plate, and left at room temperature overnight.

The three types of test specimens were prepared to show the following:

1. The relationship of varying percentage of asphaltic cement to compressive strength. (Table One, Figure One).
2. The relationship of varying percentages of mineral filler to compressive strength. (Table Two, Figure Two).
3. The relationship of varying percentages of sand in two definite asphalt/filler ratios. (Table Three, Figure Three).

According to Mr. Vokac (3), 600 pounds per square inch is considered a suitable minimum and 1,000 pounds per square inch a suitable maximum compressive strength for asphaltic paving mixtures of the hot mix type.

Referring to Figure One, at 4.61 per cent asphaltic cement, a compressive strength of 600 pounds per square inch was secured, although the test specimens were so dry that they could easily be crumbled by hand. With 12.3 per cent of asphaltic cement, another mixture, far more plastic and of good appearance, had a compressive strength of 600 pounds per square inch. Evidently compressive strength alone is not a criterion of the service behavior of asphaltic paving mixtures. Theoretically the first mixture would be expected to have a higher modulus of elasticity than the second mixture, thus suggesting that the modulus of elasticity may be a factor of consideration in studying asphaltic paving mixtures.

In order to determine how an increase in the percentage of mineral filler would affect the compressive strength, a mixture with an asphaltic cement/sand ratio of .176 was selected. As shown by Figure Two, it was found that, in general, the addition of mineral filler to such a mixture was accompanied by a rise in compressive strength. With another mixture having an asphaltic cement/sand ratio of .111, it was found that the addition of mineral filler did not bring about as large an increase in compressive strength as it did in the first case. No adequate explana-

tion of the slight drop which is present in both curves can be afforded until more data has been collected. It may be observed that a similar drop in the compressive strength also occurs in one of the graphs of Figure Three.

To show the effect a variation in the per cent of mortar* has on compressive strength, a mixture with an asphaltic cement/filler ratio of 1.00 was used. Varying the mortar content from 8 to 28 per cent brought about very little change in the compressive strength of the mixture. Another mortar with an asphaltic cement/mineral filler ratio of 0.666 was tested over the same range of mixtures. Both curves have a very small portion of their lengths rising above a compressive strength of 600 pounds per square inch, thus indicating that care should be exercised in selecting the proper amount of mortar for a mixture.

By taking data from Figures Two, Three, and the right-hand portion of One, an isodynamiagram of 600 pounds per square inch was plotted on triangular coordinate paper. The resulting graph is shown in Figure Four. That portion of the isodynamiagram containing less than 70 per cent sand is not too reliable inasmuch as part of it was obtained by interpolation; however, no mixture outside of the curve in this region was found which possessed a compressive strength greater than 600 pounds per square inch. Several mixtures, both inside and outside of the curve were tested for this information. This data is given in Table Four. It may be noted that of these mixtures, those having a compressive strength greater than 600 pounds per square inch, all fall within the enclosed area.

There is one other area on this diagram which needs further explanation. It is that area containing between five and nine per cent asphaltic cement, 79 and 85 per cent sand, and 10 and 12.5 per cent mineral filler. As has been suggested before, the compressive strength of an asphaltic paving mixture does not necessarily differentiate between a mixture containing an insufficient amount of asphalt and one containing a sufficient amount. Referring to the isodynamiagram, the two 600-pound mixtures containing 10.5 per cent mineral filler with nine per cent asphaltic cement, and 12.75 per cent mineral filler with 8.5 per cent asphaltic cement, respectively, are satisfactory from the standpoint of brittleness, whereas a mixture containing 4.6 per cent† asphaltic cement with 11.5 per cent mineral filler is unquestionably too dry to be suitable, even though it has a compressive strength of 600 pounds per square inch. Therefore, the first two aforementioned points were arbitrarily connected, as is shown on the Figure.

Considering data of Figures One, Two, Three and Four, it will be noted that (Figure One) the addition of one per cent of asphalt brings about a drop of 101 pounds per square inch in the compressive strength. In Figure Two, the portions of the curves which are above 600 pounds per square inch show a maximum change of 184 pounds per square inch with each change of one per cent in the mineral filler content. This relationship was obtained from the "AC/Sand = .176" curve. The slope

*The mixture of asphaltic cement and filler.
†See left-hand portion of Figure One.

of the other curve in this Figure is less than 18 at any place where it rises above 600 pounds per square inch.

In Figure Three, the greatest change of compressive strength per unit change of the per cent of the mortar in the area above 600 pounds per square inch is 24.5 on the "AC/Filler = 1.00" line. Of the four curves in Figures Two and Three, those containing the greater percentage of asphalt in each case are the most critical.

If Figure Four is now examined, the following observations may be made. The per cent of asphaltic cement may vary from 7.25 to 13.25, the per cent of mineral filler may vary from 10.5 to 22.75, and the per cent of sand may vary from 67 to 80.5. In terms of ratios, the asphaltic cement, mineral filler ratio may vary from a value of 3.405 to 1.000, the asphaltic cement, sand ratio may vary from a value of .1034 to .1793, and the mineral filler, sand ratio may vary from .1304 to .3034 in value. In the above named order, the per cent of variation from the mean value of the numerical range of each ratio is 49.2, 26.8, and 40 per cent, respectively. In this case, the relationship between the sand and asphaltic cement is the most critical; that between asphaltic cement and mineral filler is the least critical.

As a summary of these observations, the following conclusions may be made:

1. Variation of the percentage of mineral filler in the mixtures studied did not have as great an effect upon the compressive strength of these mixtures as a variation of the percentage of asphaltic cement.

2. A change in the value of the asphaltic cement, mineral filler ratio did not affect the compressive strength of the mixtures as much as a change of the value of the asphaltic cement, sand ratio.

TABLE NUMBER ONE

Per Cent A. C. Variable F S Constant

No.	Per Cent A.C.	Per Cent F	Per Cent S	AC/F	AC/S	F/S	Compressive Strength
1	3.5	11.58	84.92	.302	.041	.136	528
2	4.0	11.52	84.48	.347	.047	.136	584
3	4.5	11.46	84.44	.393	.054	.136	594
4	5.0	11.40	83.60	.439	.060	.136	718
5	6.0	11.28	82.72	.532	.073	.136	744
6	6.5	11.22	82.28	.579	.080	.136	833
7	7.0	11.16	81.84	.627	.086	.136	971
8	7.5	11.10	81.40	.676	.092	.136	815
9	8.0	11.04	80.96	.725	.100	.136	934
10	8.5	10.98	80.52	.774	.106	.136	977
11	9.0	10.92	80.08	.824	.112	.136	880
12	9.5	10.86	79.64	.875	.119	.136	808
13	10.5	10.74	78.76	.978	.133	.136	783
14	11.5	10.62	77.84	1.083	.147	.136	685
15	13.5	10.38	76.12	1.301	.177	.136	464

TABLE NUMBER TWO

Per Cent Filler Variable A.C. Sand Constant

No.	Per Cent A.C.	Per Cent F	Per Cent S	AC/F	AC/S	F/S	Compressive Strength
1	13.44	10.4	76.16	1.292	.176	.137	486
2	13.14	12.4	74.46	1.060	.176	.166	495
3	12.84	14.4	72.76	.892	.176	.198	615
4	12.54	16.4	71.06	.765	.176	.231	565
5	12.24	18.4	69.36	.665	.176	.265	935
11	9.16	8.4	82.44	1.090	.111	.102	483
12	8.96	10.4	80.64	.861	.111	.129	604
13	8.76	12.4	78.84	.706	.111	.157	628
14	8.66	13.4	77.94	.646	.111	.172	607
15	8.55	14.38	77.02	.595	.111	.187	478
16	8.45	15.4	76.15	.549	.111	.202	524
17	8.35	16.39	75.26	.509	.111	.218	614
18	8.15	18.38	73.47	.443	.111	.250	613

TABLE NUMBER THREE

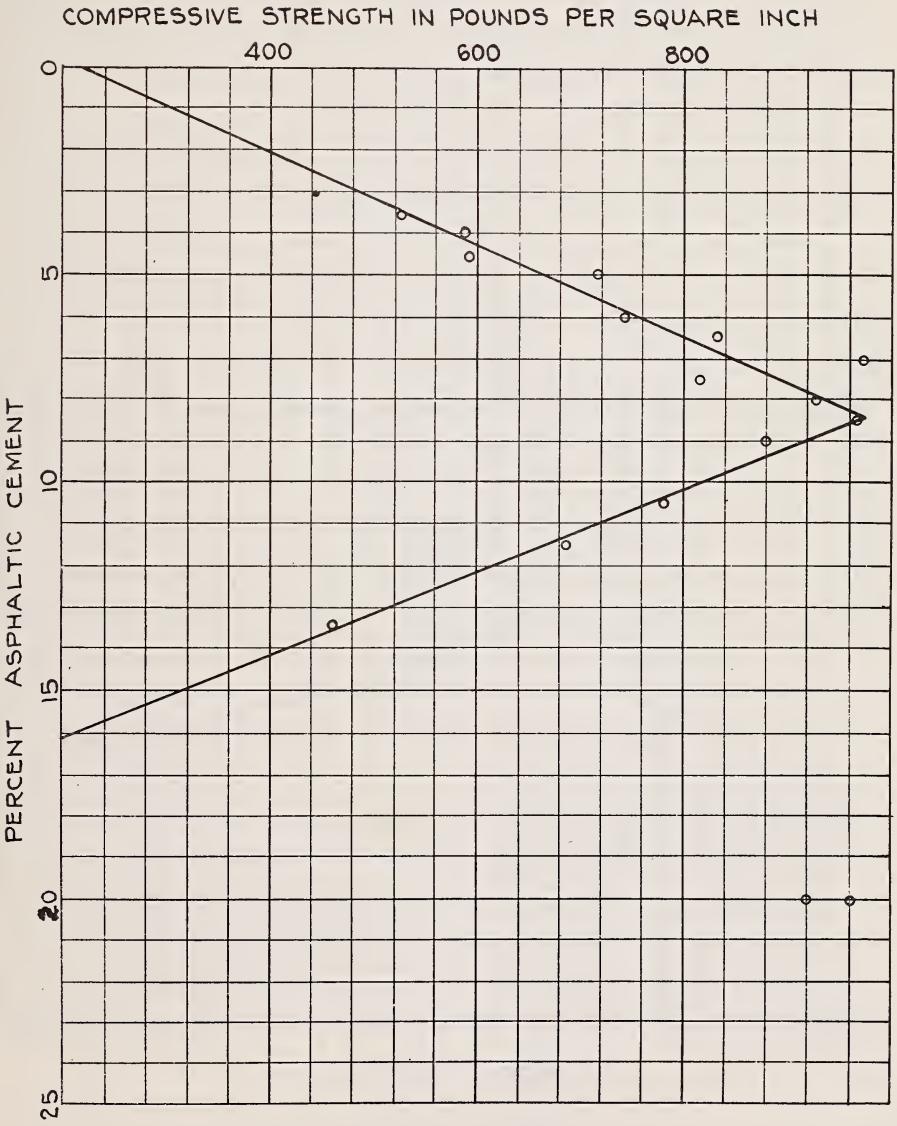
Per Cent of Mortar Variable

No.	Per Cent A.C.	Per Cent F	Per Cent S	AC/F	AC/S	F/S	Compressive Strength
1	4.5	4.5	90.0	1.00	.049	.049	456
2	6.5	6.5	87.0	1.00	.074	.074	582
3	8.5	8.5	83.0	1.00	.102	.102	561
4	9.5	9.5	81.0	1.00	.117	.117	540
5	11.1	11.1	77.8	1.00	.143	.143	615
6	12.65	12.65	77.4	1.00	.163	.163	582
7	14.15	14.15	71.7	1.00	.197	.197	173
11	3.6	5.4	9.1	.666	.039	.059	386
12	5.2	7.8	8.7	.666	.060	.090	450
13	6.8	10.2	8.3	.666	.082	.123	585
14	8.4	12.6	7.9	.666	.106	.159	605
15	10.0	15.0	7.5	.666	.133	.200	577
16	11.6	17.4	7.1	.666	.163	.245	501

TABLE NUMBER FOUR

Special Mixtures

No.	Per Cent A.C.	Per Cent F	Per Cent S	AC/F	AC/S	F/S	Compressive Strength
1	10.0	17.5	72.5	.571	.138	.241	674
2	10.0	15.9	75.0	.666	.133	.200	807
3	10.0	22.5	67.5	.444	.148	.333	849
4	12.5	20.0	67.5	.625	.185	.296	330
5	7.5	22.5	70.0	.333	.107	.321	623


COMPRESSIVE STRENGTH IN POUNDS PER SQUARE INCH
400
600
800
PERCENT ASPHALTIC CEMENT
0
5
10
15
20
25

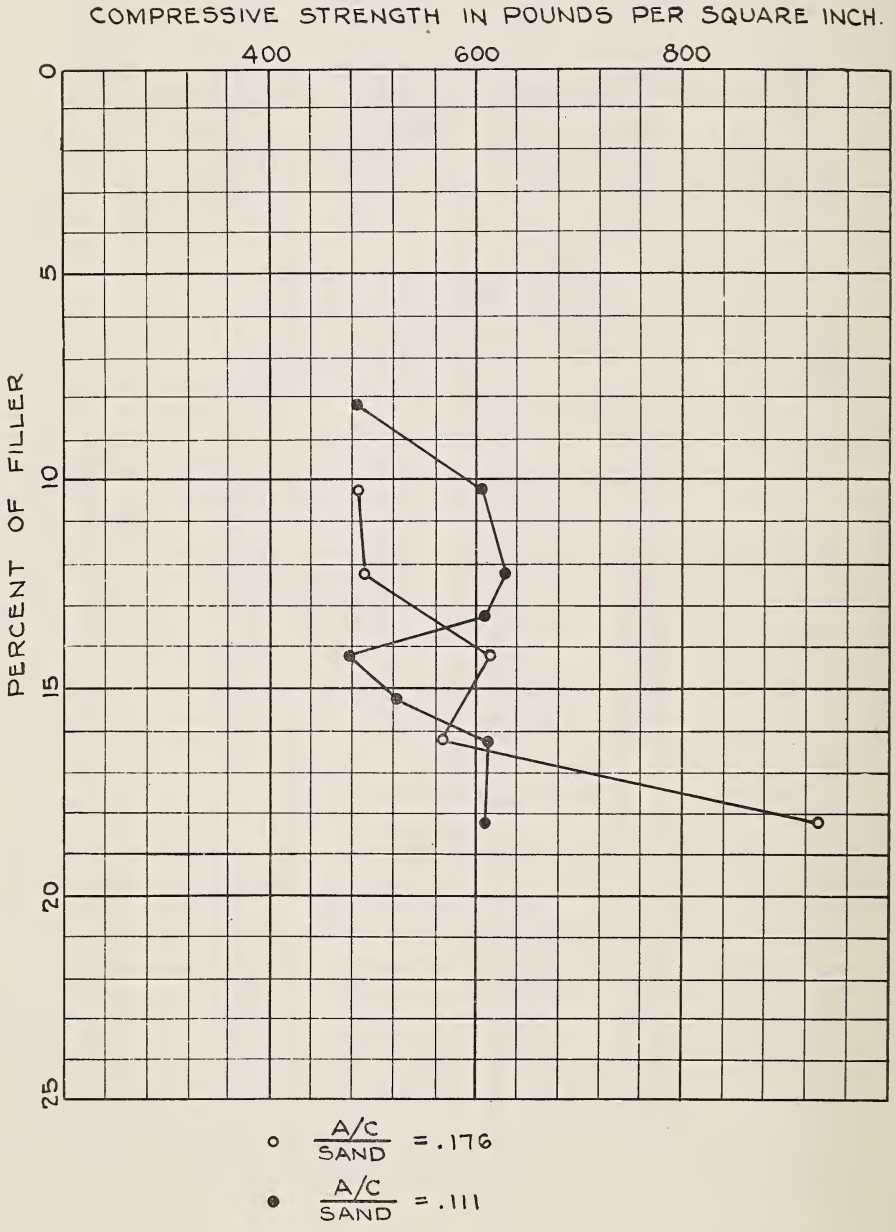
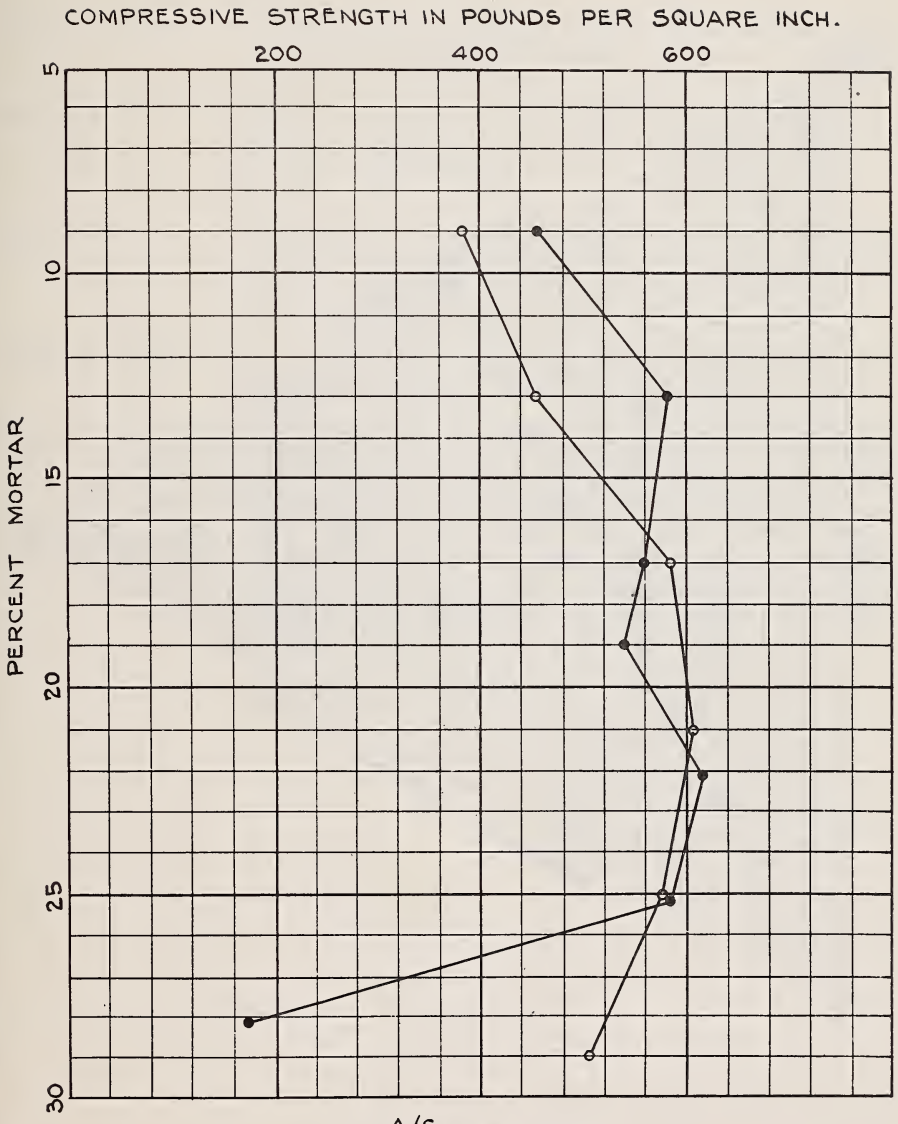

COMPRESSIVE STRENGTH IN POUNDS PER SQUARE INCH.
400
600
800
0
5
10
15
20
25
PERCENT OF FILLER
o A/C/SAND = .176
● A/C/SAND = .111

FIGURE 2.



○ $\frac{A/C}{FILLER} = 0.666$

● $\frac{A/C}{FILLER} = 1.000$

FIGURE 3.

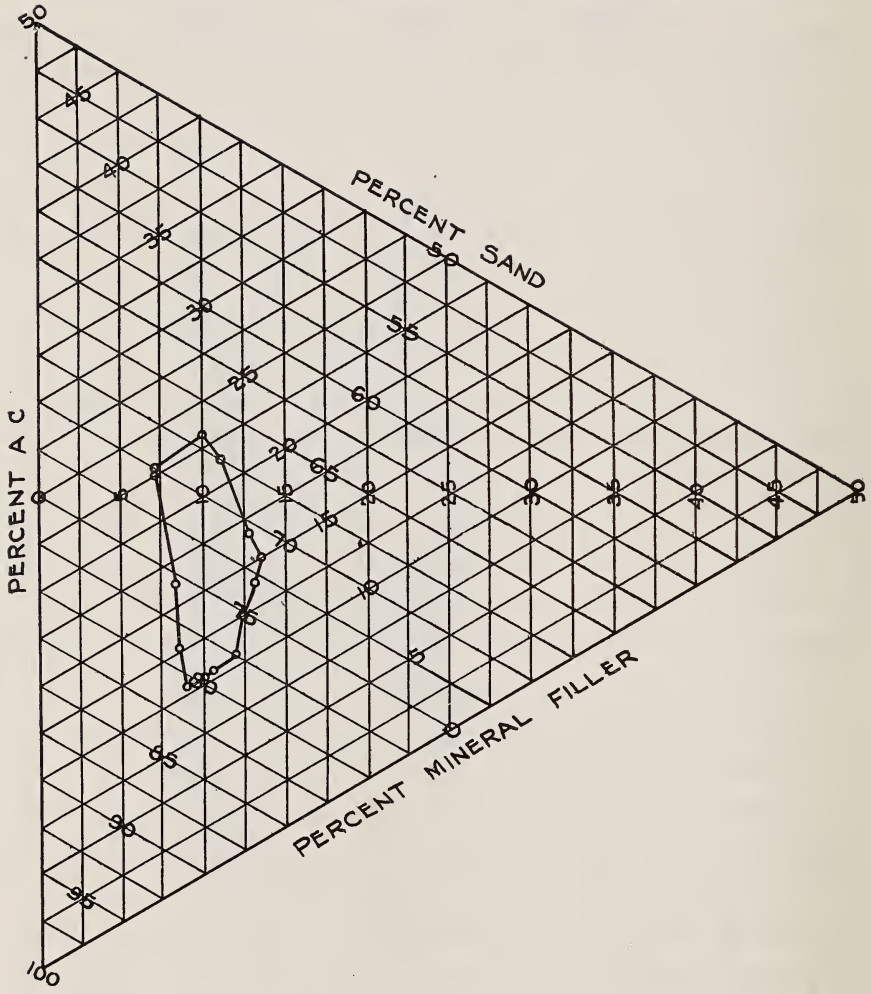


FIGURE 4.

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Factors Influencing the Selection of Seal Coat Types—When and Where to Use

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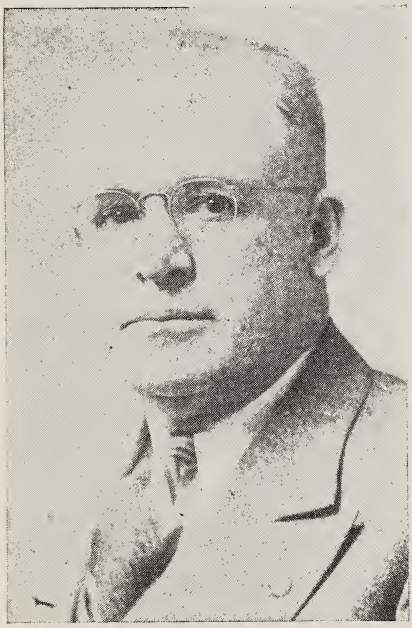

For the purpose of this discussion I will define 'seal coat'' as the last operation in the construction of a bituminous surface or in the maintenance of an old surface and it is primarily intended to protect surfaces against the ravages of traffic and the elements. Seal coat types, however, are used to accomplish various things:

(1) To prevent raveling of an open type of bituminous-aggregate surface, and to enliven and seal against entrance of moisture in a badly checked, dried out, closed type of surface.

(2) To reduce the slipperiness of an otherwise good bituminous surface.

(3) To seal, level, and make smooth riding an old rough bituminous, brick, or concrete pavement.

(4) To correct the glare of a road surface.

Although the method of procedure in all of the above classes of work may be more or less similar and some of them almost identical, there are different features which must be recognized in order to accomplish the best results. In all of them it is desirable to finish the surface with a comparatively fine graded bituminous coated aggregate that is hard and will not polish to a slippery surface.

The purpose should be to develop a durable, nonskid, smooth riding surface, free from undue glare. When it is contemplated to use such a surface only temporarily, the work naturally should be modified both in quality of material and workmanship, as well as the thickness of the layer, to economically harmonize with the proposed future plans.

While the above sets out the theoretical features to be sought, many things enter into the practical accomplishment of these results.

(1) TO PREVENT RAVELING

The most common use of the seal coat type is to seal an open bituminous surface to prevent raveling. This is accomplished by filling the voids in the surface course with bituminous coated aggregate either made in a plant or on the road, such as a surface mix or merely a surface treatment of bitumen and fine aggregate covering. Although primarily to prevent raveling of the surface, the seal coat of whatever type makes the surface more waterproof. This waterproofing is a very necessary requirement for preserving the surface. The cracking and disintegration of little used surfaces such as is true with many airport runways can be greatly avoided by better sealing them against water.

Grade of Bitumen That Can be Satisfactorily Used May Be Determined By the Amount of Traffic and Availability of Detours

Slow curing and medium curing asphalts may be more desirable for use, at least in the first applications, when building up a seal coat mat on a loose stone or gravel road. Even the medium curing products may be used on an old bituminous surface that is very badly worn and where it is necessary to build a considerable mat to properly recondition same for traffic. On secondary roads even the finishing treatment may be of the medium curing class. A slow curing and even a medium curing product is very objectionable to traffic and where, because of the lack of convenient detours or where the local traffic conditions require that the road be used quite extensively while under construction, the rapid curing asphalts should be used.

Amount and grade of Bitumen in a Seal Coat should vary with:

(a) Amount of traffic.
(b) Hardness of the aggregate.
(c) Absorptive properties of the aggregate.
(d) Geographical location.

(a) **Amount of Traffic.** The more traffic on a road, the greater precaution should be used to guard against the use of too much bitumen which may cause a "fat" and hence a slippery surface. Only the necessary amount of bitumen should be used to prevent the aggregate from raveling.

Unfortunately some of our seal coats have to be built in the late fall. More bitumen is necessary to prevent raveling in the late fall work than work done during the hot summer weather. Late fall work may require the use of so much bitumen to hold the aggregates against raveling during the winter and spring season that an excess of bitumen will come to the surface the following summer. The necessity for using an excess of bitumen in the late fall seems to be reduced to some degree by the use of emulsified asphalts, although this is not a positive cure.

Finely crushed limestone with the dust removed, coated in a plant with emulsified asphalt, may be used very satisfactorily in the late fall season for sealing a surface against raveling. In summer weather any hard grained aggregate properly coated with bitumen may be used.

Under any conditions late fall construction should be well compacted with a heavy roller to furnish the greater resistance against raveling. The

better compaction will produce a greater density and hence cause the bituminous mixture to assume the greatest stability. In this late fall construction, the going over the surface many times with a light roller is not a substitute for fewer rollings with a heavy roller. The rolling of any bituminous mixture during the proper stages of curing is most important. You can pack clays in earthen dams or around a fence post most conveniently to its greater density when the proper stiffness prevails. This feature is not always fully recognized in compacting bituminous mixtures and therefore many times the failing to recognize this fundamental principle results in second-class work.

The softer limestones show less signs of raveling when used in the late fall than do the extremely hard aggregates which are most desirable to use on a heavily traveled highway.

Old dense mixed, closed type of surfaces that are badly checked, dried out, lifeless, and on the verge of disintegrating may be given new life and their usefulness extended many years by application of proper seal coat.

When seal coat work is used to prevent raveling of a surface, it is no compliment of the work to have the seal coat itself ravel; however, it may even be better to have a small amount of raveling of the seal coat than to have much of the surface extremely rich in asphalt. This is true because of two things: first, the raveling surface can be further treated to correct it; and, second, one of the most important requirements of a modern highway is safety. If a surface is too rich in asphalt, it is not a safe one. It may be necessary for the engineer to choose the lesser of two evils when he is required to do late fall work.

It may appear that too much has been said about late fall work and the question naturally arises—why chance it? Officials of every state resolve yearly that they will not be guilty of performing late season work again but all too frequently must face a situation where they cannot deny the public use of a newly constructed road during winter months. They cannot hope to hold the surface against raveling, however, unless sealed and, since placing of seal coat is last operation in building of most bituminous surfaces, it naturally runs into late season work.

(b) **Hardness of the Aggregate.** The harder the aggregate, the more bitumen will be required; also, the harder the aggregate, the more important it is to use a heavy roller during construction, particularly during cool weather. In one instance, an engineer was known to condemn the grades of asphalt that had been used over a wide area for many years because of a grade of hard, yet porous absorptive aggregate, extensively used with insufficient bitumen (an unusual condition) which caused the bituminous surface to crack badly as it aged.

Heavy traffic will crush and polish the soft aggregate which eventually may become slippery. This is less likely to happen with the hard aggregate. It will remain dry and gritty and hence considerably more nonskid than will the soft aggregate, provided not too much asphalt is used; however, on a very light traffic road, the soft aggregate will also remain dry and therefore fairly nonskid and it will not ravel so readily as will the hard aggregate; consequently, the softer aggregate may actually have a slight

advantage on a very light traffic road where aggregates coarser than the sand sizes are used.

(c) **Absorptive Properties of the Aggregate.** Porous aggregates require slightly more bitumen than dense aggregates. Most slags, because of their porous nature will require larger amounts of bitumen; however, some limestones are also very absorptive and if the lighter grades of asphaltic materials are used may require an appreciable increase in bitumen to properly seal them against raveling. Close cooperation between the laboratory and the field forces will aid greatly in solving this problem.

(d) **Geographical Location.** Climatic conditions affect the amount and kind of bitumen that should be used in a surface mixture; also the relative amount of shade and sunshine on a surface affect the amount of bitumen that should be used. It is well known that in northern climates a softer penetration asphalt should be used; however, it is not so commonly recognized that on the shaded side of a mountain and on a roadway surface that is shaded during much of the hottest part of the day, it is also desirable to use a softer asphalt and also a slightly increased amount of same. On heavily shaded streets dense mixtures such as the sheet asphalt types are frequently more successful than the open type of mixtures; on the other hand, surfacing areas with the same sheet asphalt mix that gave good results in the dense shade may not be so successful when exposed to the maximum amount of sunshine. The more open types of mixtures usually will ravel much less in the sunshine areas.

It is readily seen that the hardness of the aggregate, its absorptive properties, the amount and kind of traffic, the geographical features, and the grade and amount of asphalt to be used in any mixture are closely related factors. It is difficult to give exact formulas or quantities for the grade and amount of bitumen because of these variables.

It is not uncommon to see a good grade of asphalt or aggregate condemned because they have not been properly proportioned. We will eventually more definitely prove that for any aggregate of a particular size and hardness there is a very limited range in the amount of bitumen that can be used for best results under certain traffic conditions.

When we take into account all of these variables, we can readily see how complicated becomes the subject of knowing how to build the most nearly perfect bituminous surface. The close cooperation of the field forces with the laboratory tests and checks, together with the observation over a period of years of the results secured with different mixes of various aggregates and asphalts, makes this problem the most approachable to a definite solution. Perhaps nothing can be better proof of the necessity of trained and experienced engineers of long service than a thorough understanding of the problems involved in this class of work.

(2) TO REDUCE THE SLIPPERINESS OF AN OTHERWISE GOOD PAVEMENT

Occasionally a bituminous surface may have developed a slippery condition. It is very fortunate for the asphalt industry that an asphalt type of road lends itself to making a nonskid surface, even though by some error it has previously been made into a very slippery one. The nonskid

character is brought about largely by the nature of the aggregate as well as by keeping the bitumen down to a minimum in the wearing surface.

Reliable tests and observation show that the fine grained hard aggregates offer the greatest resistance to slipping. The nonskid character of such a surface may be explained by the resistance the hard grains offer to wearing and polishing and to the great number of small points which contact and grip the rubber tire all at the same time.

An error not uncommon to be made is to produce a "pebbly" surface thinking that by so doing a nonskid surface has been produced. While such a surface when new and during dry weather may be nonskid, and even the new surface may be fairly nonskid when wet, a careful study shows that as such a surface wears under traffic it may become very slippery. This is the result of the corners of the surface aggregate wearing off. It is also a fact that the area of the surface in actual contact with the tire may be greatly reduced on the "pebbly" surface. A somewhat exaggerated illustration of what takes place on such a surface as it wears is shown by an old granite block pavement with the corners of the blocks worn off. Many of such pavements are exceedingly slippery.

In the surface treatment and surface mix types, fortunately, if an error has been made in adding too much bitumen, it usually can be corrected at a later date at small expense. If the surface is too rich with asphalt, a properly sized hard aggregate can be rolled into the "fat" surface on a hot day. This may be done as frequently as is necessary until the excess bitumen is converted into a dry nonskid surface.

Many of the old penetration types of road that were well built and have resisted the wear of traffic for many years have developed a slippery surface due to the polishing of the aggregate by the traffic and maybe only from a slight excess of bitumen, an amount which was really not too much when constructed to hold it against raveling. Sometimes attempts have been made to correct the slippery condition of such a surface by slightly scarifying the top layer. This is not very satisfactory for two reasons: first, the breaking through of the top crust of the old surface usually starts raveling in places and also breaks the water seal that has formed on it; second, the protection against a slippery condition is not so effective as generally believed.

A better method of correcting the slipperiness on such a surface may be to coat same with a seal coat of a nonskid mixture. While surface treatment with the proper kind of covering will produce this nonskid condition, if the proper covering aggregate is not used eventually there develops the same slippery condition as originally existed, which calls for another surface treatment. If a fine graded hard aggregate bituminous mixture is spread over the surface, the desired results will frequently be secured at the least expense.

Many of these old bituminous roads have given wonderful service and have endured for years because they have a splendid base. Such roads should never be torn up just merely to correct the slippery condition of the surface when, with the proper kind of seal coat, the difficulty is solved. With a new covering coat the old base which has already given wonderful service will be even stronger than it was before.

Sheet asphalt pavements vary greatly in their nonskid character in fferent parts of the country due to the variation in the sand used their construction. Besides the variable hardness and angularity of e sand grains, one or more other factors may contribute to their slippery nonskid character. In some places this type of pavement is so slippery at new sheet asphalt is being surface treated or given a thin seal coat of fine grained bituminous mixture of a nonskid character.

Generally speaking, the harder the aggregate the more nonskid the rface will be; also aggregates which do not polish readily but instead aintain a gritty surface are the most effective for a nonskid surface.

Inasmuch as the seal coat of any pavement constitutes a very small er cent of the total thickness of the road crust, one may be justified importing this small amount of aggregate at considerable cost to cure the most desirable and safe riding surface.

One of the principal requisites of the good modern highway is safety. he degree of safety is closely associated with the nonskid character f its surface. This means that, among other things, the surface must be s nonskid as it is practical to build it. Many engineers have failed this requirement and as a result our accident toll has not been kept own to where it might be. A few engineers are loath to give the safety actor much consideration. They will have to be classified as antiquated nd out of step with modern times.

As our highway transportation develops in amount and speed, it takes o prophet to predict that the safety factor is becoming a more important ne in our highway transportation problems.

(3) TO LEVEL AND MAKE SMOOTH RIDING AN OLD ROUGH BITUMINOUS, BRICK, OR CONCRETE PAVEMENT

The correction of the above-listed types of pavement is a class of work hat frequently involves resurfacing with more than a seal coat. This ork might be considered somewhat foreign to the subject of this paper; owever, if more than a seal coat on the old surface is required, it is sually a thin resurfacing course, the seal coat of which constitutes a arge part of the cost and hence this work seems appropriate from this oint of view for discussion under the title.

The problem here is to not only correct the roughness but to seal p the surface against water entering it which is the cause of more ailures of our pavement than many times suspected. After the depres-ions in these surfaces are patched, frequently a seal coat consisting of double surface treatment and 50 to 70 pounds of covering aggregate s all that may be required. This treatment may be repeated from year to ear as the condition of the road and amount of traffic will warrant.

If the bituminous type of road has a good base and it is merely the oughness that is to be corrected, after smoothing the surface by patching, thin seal coat of a hard aggregate either of a surface mix or plant nix may be all that is necessary.

Brick or concrete pavements usually have mobile units in their surface and, to make anything approaching a fairly stable surface again, it will

be necessary to add one inch or more of a bituminous coated aggrega either as a plant mix or a surface mix, and this should be sealed with fine grained nonskid mixture.

In sealing an old concrete or brick pavement, experience indicat that a bituminous surface treatment properly dragged in the constructio operations seals up the cracks, levels the surface, and causes it to carr traffic far better than generally suspected. It is important in this wor that a treatment of the liquid asphalt be first applied to the old surfac before any aggregate is placed thereon; of course, any pronounced weal ness or defect in the old base will have to be corrected to hold again failures again in the thawing or wet season of the year.

Inasmuch as the details of constructing such mixtures are well know it is not deemed necessary to elaborate on them in this paper.

(4) TO CORRECT THE GLARE OF A ROAD SURFACE

The same fine grained hard angular aggregate which produces th most nonskidlike surface also produces the least glare. This, apparentl is due to the diffusion of the light in all directions from the angula aggregate particles. This is in great contrast to the glaring light comin from a slick smooth surface. In other words, the same treatment tha produces a nonskid surface will at the same time produce a nonglarin surface.

Another form of glare which is very objectionable to the drive though only occasionally encountered, is that coming from an extremel white aggregate on the surface. Even though such aggregate may b tacked down with a heavy asphalt adhering to its lower surface, it extremely white surface still produces an objectionable glare in the driver' eyes. A thin coat of hard, fine grained, bituminous coated aggregate sprea over such a surface will eliminate this glare so that it will offer practicall no objections to a driver, even though he be headed toward the su at a comparatively low level.

While seal coats are in many respects very much alike in that the have for their prime purpose the sealing of the surface against ravelin and the entrance of water, and they must take all of the wear an tear of traffic, they also serve other very necessary features in a roa surface as heretofore mentioned; hence, they are a very vital part of an highway.

While many other features of seal coat work are legitimate subject for discussion, I have attempted to include herein some of those whic I believe most worthy of consideration by the designing and maintenanc engineers who are most responsible for the type of finished surfaces tha we have on the major portion of our highways.

Sub-base and Foundations for Bituminous Surfaces

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Because of insufficient revenues, highway tax diversion, and the ever-increasing requirements of modern highway transportation, it is more than ever imperative that every dollar available for road improvement be spent to best advantage, so as to provide greater safety and comfort to the public, reduce distance and time of travel and lower the cost of operation of motor vehicles and of maintenance of roads.

Alignments, curves, grades, width of shoulders and right of way, visibility by day and night, the separation of opposite and cross flows of circulation, traffic capacity, are all important factors to be considered in modern highway construction, but not more so, however, than is that of the riding surface itself.

The pavement, besides having lanes of sufficient width and in adequate number, as well as resistance to skidding and wear, should also possess a smooth riding surface which would last not only a few years following its construction but throughout its entire economic life. Such a result is a difficult task, and a cause of great concern to highway engineers.

For many years, during the time when the main consideration in road construction was the horse-drawn vehicle, water bound macadam surfaces were believed to be of sufficient resistance to last a long time. With the advent and subsequent increase in the number of motor vehicles this belief was soon dispelled, and more resistant surfaces in particular, those of the bituminous type, were built with the confidence that they would not be torn up by this new type of locomotion.

However, numerous failures developed on these pavements which could not be attributed to their use by motor vehicles and which, after investigation as to their cause by the Bureau of Public Roads of the United States and other organizations, led to the discovery that subgrade soils possess certain significant characteristics. These soils react accordingly under particular conditions and, in doing so, they disturb and damage the pavement and even destroy it in many cases. Further investigation and studies of the properties of soils and of the means of overcoming their destructive reactions have led to this new science of soil engineering as it is known now, but which is yet at its beginning and of which still greater things are expected.

Today, it is generally admitted that a pavement is as good as the base that supports it and no better and that all efforts and expenses made to built a durable and smooth riding surface are futile if these unfavorable

reactions to which the subgrade is subjected are not investigated and the proper means to counteract them are not taken.

The purpose of this short paper is not to discuss the engineeri properties of soils about which information has recently been assembled text-book form by Mr. C. A. Hogentogler. It is simply intended to rec briefly what are the most important disturbances occurring in the su grades and what are some of the means known to date to prevent the or at least reduce their magnitude.

MOST COMMON CAUSES OF PAVEMENT FAILURES

Amongst the most common causes of pavement failures, which ma be attributed to the instability of the subgrade, are the following:

1. Plastic deformation under traffic load and impact, particular in cuts and shallow fills and volume change whenever unfavorable sc and moisture conditions exist.

2. Frost heaving due to the deformation of growth of ice laye in the subgrade with subsequent collapse of foundations and pavemen

3. Settlement of fills, which may last for many years and may b quite irregular, due to the insufficient and uneven compaction at tim of construction.

4. Rebound in subgrade and expansion in fills after attempt ha been made to compact soils possessing elastic properties and a certai degree of cohesion.

5. Sliding of side slopes in cuts and fills due to an excess of moistu in certain soils; insufficient slope being given or sliding over an impervio layer of soil underneath.

These are some of the causes of the distortion, the deterioration an even the destruction of many pavements which had been laid with car so that their surface might be as nearly perfect as possible. To protec them against such deterioration can only be done by getting at the roc of the evil, which is now possible, thanks to this new science of so engineering.

Subgrade Soils Stabilization

Although much has yet to be learned about more economical mean of stabilizing subgrade soils, sufficient advances have already been mad to be a great help in doing so. The means presently available are know to most highway engineers but, nevertheless, a brief review of them i always advantageous.

Any efforts by road building organizations to improve subgrade stabilit should first be directed towards the training of specialists for both fiel and laboratory work, so that investigation of soil conditions may b made adequately and so that proper remedies be applied where needed

Location and Profile

On new construction where troublesome soil conditions are detected the possibility of changing the location of the road should be studied firs of all and, in many cases, it will be found possible and economical to d so without prejudice to other important standards of the highway.

Once the proposed road has been located definitely and a tentative ·ofile chosen, study should then be made of soil and moisture conditions ong this profile and, where it is found advantageous to do so, the profile ould be modified to best advantage.

Water Most Disturbing Agent

After both the most suitable location and profile have been chosen, her unfavorable soil conditions still remain to be dealt with which nnot always be easily and economically remedied.

The extent to which remedial measures may be applied depends upon ıe importance of the road and the kind of surfacing to be built although ı many cases the nature of the subgrade itself should influence the ıoice of the pavement.

When large structures are built, it is necessary, because of the heavy)ad concerned, to erect them on very strong foundations. In the case f roads, however, the loads to be carried are small in comparison so ıat most soils, in their dry state, have nearly enough supporting power) resist to the loads and shocks of traffic and are not liable to suffer ny reactions that may cause serious disturbances in the subgrade.

In presence of an excess of humidity, it is not so, however, and soils re affected in various manners according to their characteristics. Their upporting strength may be so reduced that they may become quite un-table and upon freezing and thawing they may cause disturbances so reat as to damage and even ruin the best of pavements. Alternate vetting and drying produces very detrimental volume changes in ex-)ansive soils.

Since water is the cause of the most important reactions to which ubgrade soils are susceptible, it is therefore towards the control of this ıost disturbing agent, which may reach the subgrade either from the urface or the subsurface, that the greatest attention should be directed.

Surface Drainage

In dealing with surface water, it is important to make sure of its rapid ınd complete evacuation, at all times, from the surface of the pavement ınd shoulders towards the side-ditches and, therefrom, towards the cross ditches or streams.

For this reason, the pavement must have a sufficient crown and the shoulders a regular slope toward the ditches. Shoulders in particular should never be allowed anywhere to act as catch basins for surface waters. Culverts and bridges must always be kept in good working condition as well as side ditches which should be protected against erosion or sedimentation.

Special attention must be given to surface waters in late winter and early spring when snow and ice hampers its rapid evacuation from shoulders, side-ditches and culverts. Openings must be provided wherever needed and as early in these seasons as required.

Shoulders, particularly when covered with grass, may allow enough water to penetrate the roadbed to cause serious disturbances therein. Treatment of these shoulders to make them impervious will generally be found of great benefit and sometimes essential.

SUB-SURFACE DRAINAGE

If the control of surface waters to prevent them from reaching t subgrade is relatively easy, the same is not true of sub-surface wate

Such waters may be present under three forms: gravitational whi can be drained; capillary which cannot be drained and hygroscopic of much less serious nature.

(a) **Gravitational Waters.**

Sub-drainage of roadbeds to prevent gravitational or free waters flow into them has been attempted for years but, very often, with little success that many engineers doubted that efficient drainage wa possible and so resorted to heavier foundations and thicker pavements obtain a more stable surface, only to find these foundations partly gon down into the subgrade in many instances as well as the pavemen damaged or ruined only a few years after their construction.

In recent years, however, because of the use of better materials an type of drains and because especially of a better knowledge of soils, ver efficient sub-drainage is possible.

Many factors have to be considered in this work, such as the nature c the various soil layers and their form; the character of the soil moisture the elevation of the water table; the topography of the region and th climatic conditions.

Soils vary in permeability according to the relation of their pore and solids, and the rate of percolation through a soil depends upon th permeability. It follows therefore that the movement of gravitational wate in the ground is to a large extent controlled by the degree of permeabilit of the various soil layers and their formation.

In hilly or rolling country, gravitational water reaches the roadbe by the lateral movement it is forced to take when the percolation rat of a soil layer is lower than that of the one above it.

Sub-drain built of porous materials can therefore be used to intercep free water moving laterally in the subgrade and by means of a pipe t which water may have access, this water can be led by gravity to a prope outlet.

In order to intercept or catch all of the water flowing laterally o resting above an impervious soil layer, drains must be built down to tha layer. When this impervious layer is too far below the surface, it is no possible, and perhaps not necessary in some cases, to build the drain a far down but, even then, the elevation of the water table may be lowered enough to suit the purpose.

Knowledge of the characteristics of the various soil layers as well a their form, and knowledge of the direction in which gravitational water flow as well as the elevation of the water table is therefore essential i sub-drainage design.

When the flow of the underground water is transversal to the axis of the road, sub-drains must be placed alongside the roadbed facing the

ow of water. When the flow is along the axis of the road, the drain must placed diagonally across the road somewhere above the point where e seepage zone and the subgrade surface meet.

If water is trapped in an impervious soil pocket filled with a porous il a drain must be built to free this water from under the roadbed.

In flat or swampy country where the water table is level, with little no lateral movement, but where the water table elevation varies with e amount of rain precipitation, sub-drainage is difficult and generally npossible due to lack of outlets.

Under such conditions, the possibility of lowering the elevation of the ater table by open ditches should be investigated. Where the water table evation may be lowered to a certain extent by means of sub-drainage e depth at which they may work must be determined. A drain on both des of the roadbed may be worth the extra expense.

(b) **Capillary Moisture.**

Capillary moisture is that which may move in all directions into a il whose particles are small enough, and which cannot be drained because ie forces involved are greater than that of gravity.

The finer the soil particles the higher the capillary rise, and the arser they are the greater is the rate of flow.

The fact that there is a limit to the capillary rise of moisture above ie level of the free water supply for each kind of soil makes it possible control to a certain extent the height of the capillary fringe. If, there-re, sub-drainage cannot remove capillary water, it is nevertheless useful lower the capillary fringe by lowering the elevation of the water ible.

Excess capillary moisture reduces the bearing power of soils and hen such conditions exist close enough to the road surface, plastic de-rmation of the soil may take place under traffic and detrimental volume anges may occur. Capillary moisture is most objectionable, however, hen it supplies the water necessary to the growth of ice layers in the oadbed.

Frost Heaving

In regions of low temperature in winter, the most frequent and erious damages to pavements are due to the freezing of capillary water articles in the larger soil pores which draw to themselves the non-reezing water particles of the smaller pores. As new water particles re drawn and frozen, due to a free water supply below, the roadbed nd pavement are lifted to make room for the growing ice layers and re there seriously damaged.

In the spring, upon thawing of the roadbed and ice layers, the excess f water present, which cannot escape through the still frozen soil layers nderneath, turns the subgrade into an almost liquid mud. Under traffic, specially, the foundations are dislocked and sunk into the roadbed while he pavement is damaged and eventually ruined. Closing of the road to raffic during the thawing period protects the road considerably but s of great inconvenience to the public.

To prevent or reduce frost heaving, study must be made, whenever t is liable to happen, of the best remedial measures to be taken.

The first remedial measure to be considered is that of sub-drainage by means of which the free water supply may be intercepted or the water table lowered sufficiently to bring the capillary fringe below the frost penetration.

When it is found impossible with sub-drainage to lower the capillary fringe out of reach of frost penetration, or enough to reduce frost heaving appreciably, other possible remedial measures must be studied.

One of these measures is that of raising the roadbed over the total thickness of frost penetration with suitable materials such as sand, gravel or stone. This, however, would often be too costly so that, for reasons of economy, these materials are generally laid over a less thickness with very good results when, by doing so, the capillary fringe is kept entirely or even partly, below the frost penetration depth. In recent years, it has been the practice in the Province of Quebec to lay a sand cushion, eight to twelve inches thick, over the whole width of the subgrade on all important new construction. These sand cushions, in some cases however, have not proved to be a sufficient provision against damages by frost which reaches from three to four feet in the ground.

When a layer of sand or of any granular material is thus used, it is important to provide adequate drainage to it and also to prevent, by suitable means, water entering it from the surface as, otherwise, such a sand layer might prove to be very detrimental rather than beneficial.

Another measure consists in placing an impervious layer in the roadbed, a bituminous one for example, so as to cut off the capillary rise of water into the total or partial depth of frost penetration. The capillary rise may also be cut off by means of a layer of soil of low capillarity whose thickness must be greater than the capillary capacity of the material used.

Settlement of Fills

In the building of fills the compressibility of the material used as well as that of the soil supporting it is a matter requiring due attention especially because of the present needs for higher embankment in many places.

Fills which have not been compacted sufficiently nor uniformly during construction will continue to settle, sometimes for many years, and thus cause disturbances in the subgrade and damage to the pavement.

When granular materials are used a higher degree of stability may be obtained; compaction is more rapid and settlement smaller. Any mechanical device that will apply sufficient load or pressure, such as flat wheel rollers, tractors and tamping machines, will consolidate the soil downwards over the whole thickness of each layer and compaction at optimum moisture content is not essential.

When plastic soils are used they are loosened and contain a large number of clods after being excavated from cuts or borrows. Settlement is considerable and slow, so greater care must be taken in the construction of fills made up of these materials. The stability of such fills depends on the densification of the soil which cannot be obtained by merely applying pressure upon them. The soil, placed in small layers of ten to twelve inches in depth, must be compacted from the bottom up by means of appropriate machines such as the sheeps foot roller whose prongs penetrate into the soil and thus reduce the larger voids to a minimum.

However, this is not sufficient, and so that these soils may become table enough, it is also necessary that the smaller voids be reduced by compacting the soil at its optimum moisture content (approximately equal o its plastic limit). Thus the particles will be lubricated sufficiently to lide into a position occupying the least space possible. A moisture content vhich, during compaction, exceeds the ideal one, not only prevents proper lensification because it fills the soil pores but also decreases the stability of the soil and upon drying will cause the soil to shrink and crack, and his very appreciably in some cases, thus facilitating penetration by vater.

Elastic soils should not be used in the construction of fills. Such soils vhen possessing a certain degree of cohesion may remain compressed to a certain extent when pressure is applied to them, and later, in presence of water, they will expand thus reducing the stability of the fill and leformation of the pavement.

Swamps

When, for some reasons, it has been found impossible to change he location of a road so as to avoid an area of inconsistent soil, what nay be done then to obtain a stable subgrade depends a great deal on he importance of the road, the depth and moisture content of the soil nd the height of the fill to be built.

The possibility of economical drainage of the area should first be studied. A reduction of the moisture content may often increase the stability enough to permit the construction of a fill over the natural soil.

Removal in all cases of the inconsistent soil over the entire depth, nd its replacement with suitable material is of course greatly preferable ut not always possible in all cases for measures of economy.

Swamps of shallow depths, say 5 feet or less, made up of a very nconsistent soil, should be excavated, removed and replaced with a suitable material. If, however, it is not too inconsistent and if covered with a strong meadow mat, a fill may be built over it, the weight of which will cause the soil underneath to flow gradually until a relative state of equilibrium is reached after which, only, the pavement should e built. When the weight of a fill, eight feet or more in height, is not sufficient to displace the underlying soil, trenches may be dug on each side and filled with sand, gravel or stone to prevent future settlement.

In swamps more than about 5 feet in depth, the soil must be removed by the most economical means. The weight of the fill sometimes may be sufficient to displace the soil underneath, but otherwise it must be excavated. When of greater depths, it may be displaced by means of explosive.

In all cases of materials partially or totally displaced, the fill should consist of very stable materials, sand, gravel or stone at least to a height above the ground level, exceeding the capillary capacity of the material used.

Sliding of Side Slopes of Cuts and Fills

Another cause of damages to roadbed and pavement is the sliding of side slopes of cuts and fills.

Embankments made of fine materials which contain a large amount of moisture when being built or later soaked by rainwaters have a tendency to flow laterally. Fine sands with a small amount of clay or silt can absorb

enough water to become very susceptible to such flow and may start to slide, particularly under vibration produced by traffic. Such sliding may also occur in cuts.

The kind of material used and moisture conditions are not always the cause of such slidings which may be due to insufficient slope being given to sides of cuts and fills.

For non-cohesive soils the natural slope of the material may prove sufficient whatever the height of the fill, but for cohesive ones the slope to be given must suit the particular soil used and the height of the embankment or trench. The greater the height, the flatter the slope must be.

Sliding of side slopes in cuts may also be caused by a lubricated surface of soil underneath which can be remedied by sub-drainage, after a close study has been made of soil and moisture conditions in each particular case.

Foundations

The review just made concerns chiefly some of the most common causes of pavement failures due to subgrade soils reactions. But roadbeds even when protected against such reactions, are not as a rule strong enough to offer adequate resistance to the load and shocks of modern traffic which is transmitted to it through the pavements. The variation in the supporting power of subgrade soils, from one place to another requires that an adequate factor of safety be provided so that the smooth riding qualities of the surface may be preserved.

This supporting power may be increased by stabilization of the fine grained or poorly graded soil of which the subgrade is usually composed. This may be done by admixtures of either bituminous materials or portland cement, and by blending of the soil with coarser aggregates such as sand, gravel, quarry waste, or other granular materials which permit a reduction in the amount of stabilizer used.

When well graded materials are available at reasonable cost it is preferable to build a foundation of gravel, or crushed stone, which is made more stable by adding and proportioning the quantities of frictional and cohesive materials that may be required. Such a foundation when compacted or densified with admixtures of a stabilizing chemical, such as sodium or calcium chloride, will distribute the load and shocks of traffic over a much wider surface than a non-stabilized layer of the same thickness. According to Mr. Gray, chief engineer of the Asphalt Institute, a stabilized base of six inches is unquestionably superior to a non-stabilized one eight (8) to ten (10) inches thick.

Stabilized bases also have the great advantage of providing an impervious layer over the roadbed against the penetration of surface water; of permitting earlier construction of the pavement and of facilitating circulation over the road when detours are difficult. Weak spots in the subgrade which may have been overlooked during construction are more easily detected before surfacing when the base has been stabilized.

Conclusion

In conclusion, it may be said that every country, state or province has subgrade problems of its own, which differ according to soil, climatic and other conditions but, whatever these problems may be, their solution lies on subgrade investigation and stabilization based on the knowledge of the engineering properties of soils.

Durability of Asphaltic Binders

By

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At the 1937 meeting of the Montana National Bituminous Conference at Glacier National Park, Mr. Skarin, of Alberta, made a statement [1] of utmost significance to producers and users of asphaltic road materials. He pointed out that he had been connected with a Paving Company which has built pavements continuously since 1905, and that in the early days they were able to lay sheet asphalt and asphalt concrete pavements without many cracks occurring. "In recent years," he stated, "it seems next to impossible to lay those pavements without cracks, using the same methods and materials, the only change being that we now have better equipment." He went on to say, "I have noticed pavements laid 20 and 25 years ago, and feel rather ashamed to come back to the pavements laid 4 or 5 years ago and see how much more they are cracking than those of 25 years ago." Later Mr. Skarin stated that the asphalts are not from the same sources as those used in earlier pavements.

Mr. Benson, of Kansas, followed with a discussion [2] of pavements laid in 1911 and 1917. He said that the sheet asphalt pavement laid in 1911 is still in excellent condition, and that the asphalt recovered by extraction now has a penetration of 21 and a ductility of 150+. He said that the second sheet asphalt pavement, laid in 1917, was badly cracked and had large areas requiring replacement in the winter of 1936 and 1937. A sample of the asphalt extracted from this 1917 pavement shows only 11 penetration and 4 cm. of ductility. He made the further comment: "The mats constructed of MC cutback material were very hard and soon cracked badly. The 1936 extractions on eight 1935 construction projects in which thirteen extractions were run, gave asphalts from 45 penetration in two samples to as low as 13 penetration in one sample. The age of the materials extracted was 10 to 14 months. . . . These cutbacks originally had a penetration of 109 to 151. The highest penetration, that of 151, dropped to 19 at the end of 12 months."

These comments, one coming from a capable practical paving man with many years of bituminous paving experience, and the other from an able

young asphalt technician who has done some splendid independent re search work, are chosen as being typical expressions of the uneasines which at this time is felt amongst those closely allied with the pavin and road-building industry, — both by producers and users.

The first fundamental in the construction of any pavement or roa surfacing is that the asphaltic material must be suitable, —not necessaril merely suitable at the time of use, but durable and capable of functionin as an effective binder during the expected life of the pavement.

With the thought of promoting discussion and constructive criticism, th following items are presented as considerations of vital importance in bitumi nous road construction:

1. A bituminous road may fail to give expected service because of unsuitable aggregates, or filler, inadequate subgrade or base, or improper methods of construction, but no bituminous road can be durable if the asphaltic binder does not retain its ductility in the pavement.

2. The binder left in the road is the residual asphalt remaining after evaporation and oxidation of the lighter fractions, if any, and the continued oxidation of the residuum remaining as a coating on mineral particles, whether the road be constructed with hot asphalt, emulsified asphalt, cutback, or road oil.

3. A brittle residual asphalt cannot produce a tough, flexible road surfacing.

4. Painstaking attention has beeen given to physical tests of bituminous binders at the time of use and generally very little attention to the quality and durability of the binder which remains in the road after exposure to oxidation.

5. Many of the usual specification requirements are identification tests adopted by the user and represent merely one or more of the properties of some particular binder with which the user has had success. Usually, the success in the pavement was due to some other property not known or understood, and the identification tests were thereafter included in an effort to get the same asphaltic binder. Identification tests are readily overcome by skillful manufacture even with unsuitable crudes, and such tests are, therefore useless and misleading.

The standard tests for bituminous material are usually only measures of the consistency, (penetration, viscosity, float test, etc.), of the ductility at the time of manufacture, and of solubility. If a proper asphaltic crude has been used, these tests do, to some extent, insure proper manufacture. Skillful technicians, however, can produce the same physical properties from non-asphaltic or semi-asphaltic crudes and from such crudes, these test methods merely insure usable asphaltic binder without reference to its durability.

6. It is easily possible to make poor asphaltic binders from good crude, but it is extremely doubtful if really good asphaltic binder can ever be made from unsuitable crude petroleum.

7. Asphaltic binders intended for use cold, or at low temperatures, such as cutbacks, road oils and asphaltic emulsions, cannot be better than the asphalt from which they are made. Solvents in cutbacks, upon evaporation or oxidation, do not build up body in the asphalt, or add to its ductility. They may even hasten oxidation and brittleness. Heavy oils from suitable crudes, whether in road oils or in light asphalt, may add to the quantity and quality of the residual asphalt upon oxidation.

8. Standard tests plus new test methods described in recent technical literature, with some possible modifications, can be made to show the suitability of a binder, before its use, and give reasonable indication of its durability.

9. Soft asphalts, if properly made, have longer useful life than hard asphalts. The required stability can be obtained by the use of thinner films with soft asphalt, and will result in longer life than can be obtained from hard asphalt in thicker films.

10. Ductility is the most important physical property of asphalt or asphaltic residuum recovered from pavements. Penetration of the asphalt, or of asphaltic residue, is relatively unimportant, except that low penetrations are frequently indicative of low ductilities. The old theory that it was necessary to use hard asphalt in order to secure stability is entirely erroneous, and has been the direct cause of extensive pavement failures.

The Aging of Asphaltic Materials in Road and Street Surfaces

It has long been known that asphalt hardens and becomes more brittle as it ages in a pavement, due to air oxidation. Hubbard and Reeves, in 1912, gave a very interesting paper[3] which showed a native asphalt hardening from 51 to 12 penetration during 12 months of outdoor exposure in a box with a plate-glass cover. An oil asphalt, in the same test, dropped from 228 to 55 penetration and a petroleum residuum which was too soft for a penetration at the beginning of the test, had a penetration of 73 after 12 months.

No reliable method of extracting and recovering asphalt from pavement without greatly changing its properties, was recognized until the publication, in 1933, of the Abson Test Method[4]. Lacking means of recovering asphalt without seriously altering its properties, most paving engineers were not fully aware of the rapid hardening which occurs during the mixing of a hot pavement, or that early cracking in pavement is frequently due to the rapid increase in hardness and loss of ductility of the asphaltic materials in the pavements under traffic, or that many asphalts are susceptible to extremely rapid deterioration of this kind.

Since a tool for recovery has been provided engineers in the form of the Abson Test, more progress has been made in studying the aging of asphaltic materials in road surfaces than in all preceding time. Many splendid research papers have been written, some of which will be referred to herein. It is now generally recognized that cracking in pavement is frequently due to air oxidation of the asphalt, in hot mixes at the time of mixing, and in all pavements after laying. It is also known

that different asphalts perform differently, some oxidizing rapidly and others very slowly. While most of the papers already presented deal with paving asphalt in hot mixed pavements, the same problem exists in asphaltic surfacing where cold liquid binders are used. Note again the comment by Benson[2] quoted in the second paragraph of this paper.

The usual specification physical test requirements give little or no indication of future performance of an asphaltic binder. Zapata[5] points out that when two oils from different sources were used on adjacent sections of one road, one oil produced a mat that remained well bonded, and could be lifted in chunks, and the other oil produced a mat that was loose brown and crumbly. Both oils met the same specification tests. Traffic, aggregate, weather and subgrade conditions were identical. This observation has, doubtless, been duplicated by every engineer experienced in asphaltic construction.

Research Methods and Studies

Several methods of determining the effect of aging on asphaltic materials have been proposed by our leading asphaltic technicians and results obtained have been described in their recent papers. Skidmore and Abson[6], Hubbard[7], and Steinbaugh and Brown[8], use the Abson Method of recovery and make tests on asphalts recovered from pavement and from laboratory mixes after heat treatment. Nicholson[9] describes an air blowing test, and Raschig and Doyle[10] describe a similar test with different apparatus. Benson[11] describes a splendid study of oxidation of asphaltic materials spread in thin films on glass slides under different methods of exposure. Raschig and Doyle[12] also propose flexural strength tests for specimens sawed from pavement and molded from laboratory mixes. Zapata[5] reports a study of the changes in the composition of asphaltic materials due to weathering.

All of these papers apparently lead to the same general conclusions, probably the most important of which are:

1. That asphaltic materials harden when exposed to air and/or light.
2. That asphaltic materials also decrease in ductility when aged in pavement or subjected to accelerated oxidation tests. (No definite replationship is shown between penetration and ductility.)
3. That the differences in performance of the asphalts may be due to differences in the crude from which the asphalts are produced or from different methods of refining, or both.

Tables

Although much valuable data has been presented in the many papers already referred to, and these data will be mentioned later in this paper, ten additional tables are here included.

The Asphalt Division of the Department of Public Works of Detroit, Michigan, under the direction of Clarence A. Proctor, has done extensive research work on oxidation during recent years, and the writer has been given premission to use some of their data. Grateful acknowledgment is here made to Mr. Proctor, Mr. C. L. Shattuck, in charge of the Bituminous Laboratory, and to Mr. R. G. Cobb, who assisted in the work.

TABLE 1

DECREASES IN PENETRATION AND DUCTILITY

(2,000 Gram Samples)

ASPHALT CONCRETE: 56% ROCK, 31% SAND, 6% ASPHALT MIXED AND HEATED at 338°-356° F.

Source of Crude	PENETRATION AT 77° F. Orig. Pen. Asphalt	Minutes Heated 30	60	90	120	DUCTILITY AT 77° F. Orig. Duc. Asphalt	Minutes Heated 30	60	90	120
Texas	56	43.5	36	26.5	25	150+	54	20	7.5	6
Mexico	51	43	36	31	27	150+	54	20	7.5	6
Mexico	51	36	31.5	24	25	146	42.5	22	7.5	7.5
California	67	51	44	24	24	100+	100+	100+	100+	100+

TABLE 2

DECREASES IN PENETRATION AND DUCTILITY

(1,200 Gram Samples)

SHEET ASPHALT: 75% SAND, 15% FILLER, 10% ASPHALT

MIXED AND HEATED AT 375° - 400° F. IN LARGE OVEN AND STIRRED EVERY 15 MINUTES

Source Of Crude	Specific Gravity	PENETRATION AT 77° F. Orig. Pen. Asph.	Minutes Heated 30	60	90	120	DUCTILITY AT 77° F. Orig. Ductility Asphalt	Minutes Heated 30	60	90	120
Texas	1.020	51	25	20	17	13	100+	8	7	3	2
Texas	1.020	52	29	23	21	16.5	100+	4.5	3.5	3	2.5
Mexico	1.044	55	49	27	27	21	100+	15	7	7	5
Mexico	1.044	55	35	31	23	18	100+	33.5	16	5.5	4.5
Mexico	1.043	53	35	24	21	17	100+	22.5	7	5	4
California	1.013	67	33	20	16	16	100+	100+	100+	15	17.5

Note: In these tests penetrations all decrease similarly in degree, but ductilities do not.

Tables 1 to 4, inclusive, consist of data furnished by Mr. Shattuck. Table 1 shows decreases in penetration and ductility of asphalt extracted by Abson Method from laboratory-made asphaltic concrete, due to maintaining mixtures at an elevated temperature for periods of time corresponding to the time necessary, under varying conditions, to mix, haul and place asphaltic mixtures.

Table 2 shows similar tests with sheet asphalt mixes. These tests show serious decreases in ductility as a result of heat treatment, and since 1936 Detroit has made Abson Recovery tests on all asphaltic mixes laid in that city. The ductility and penetration decreases shown in Tables 1 and 2 were found to be similar to those which occurred in the regular plant mixtures. As a result, the City now specifies and uses asphalts which do not show excessive decrease in penetration and ductility during mixing and placing, or after the pavement is laid.

Studies were made of existing streets in Detroit and the present ductility and penetration of the asphalt determined. The result of this study is shown in Table 3, in which three 21 to 26-year old pavements were found to contain asphalt with ductility (at 77° F.) of 10 to 22, while 6-year old badly cracked pavements showed ductilities ranging from 4 to 21.

Mr. Shattuck devised oxidation tests to simulate in effect the aging

of asphalt in pavement. Method "A" uses Ottawa Sand (20/30 mesh) t eliminate dust and filler in the extracted sample. Method "B" run fc comparison, uses a sheet asphalt grading. The two tests are made a follows:

"A" Ottawa Sand Mix.

(1) Aggregate 3,700 grams 20/30 mesh Ottawa Sand.
(2) Asphalt 300 grams Asphalt Cement (7.5% AC).
(3) Mixing Temperature 325° F. to 350° F.
(4) Mixing Time — 3 minutes (hand mixed with spatula).
(5) Sample split into two portions of 2,000 grams each, placed in shallow pans and placed in electric oven for one hour. After time has elapsed, the mixture is removed from oven, stirred for two minutes, the bitumen then extracted and recovered, using Abson Method. The recovered bitumen then tested for penetration at 77° F. and Ductility at 60° F. and 77° F.

"B" Sheet Asphalt Mix.

(1) Aggregate 3,000 grams Asphalt sand graded as follows: (75% of total mix).
Passing 8 mesh, Retained 40 mesh 22%
Passing 40 mesh, Retained 80 mesh 50%
Passing 80 mesh, Retained 200 mesh 25%
Passing 200 mesh, ... 3%
(2) 600 grams Solvay Limestone Dust graded as follows: (15% of total mix).
Passing 50 mesh 100%
Passing 200 mesh 85%
(3) Asphalt 400 grams. (10% of total mix).
(4) Mixing Temperature 325° F. to 350° F.
(5) Mixing Time — 3 minutes (hand mixed with spatula).
(6) Procedure same as the Ottawa sand mix.

TABLE 3

DUCTILITY AND PENTRATION OF ASPHALT IN SHEET ASPHALT PAVEMENTS IN DETROIT

Asphalt Recovered by Abson Process*

Age at Test	Condition at Time of Test	Source of Asphalt	% Asphalt Recovered	% 200 Mesh	Penetration at 77° F.		Ductility at 77° F.	
					Orig.	Present	Orig.	Present
26 Years	Good	Calif.	9.90	7.5	60.7	23	110+	10
22 Years	Good	Calif.	9.70	8.7	Unknown	10	110+	18
21 Years	Good	Calif.	9.70	9.8	Unknown	25	110+	22
6 Years	Badly Cracked	Mexico	9.5	9.4	41	30	110+	21
6 Years	Badly Cracked	Mexico	9.7	8.5	41	12	100+	4
6 Years	Cracked	Mexico	9.1	7.9	40	16	110+	4
6 Years	Aver. 16 Badly Cracked Streets	Mexico			41	24	110+	14

* Tests by Proctor and Shattuck, Detroit City Asphalt Laboratory.

TABLE 4

OXIDATION AND RECOVERY TESTS ASPHALTS FROM VARIOUS SOURCES

February, 1936. **City of Detroit Paving Laboratory**

Lab. No.	Kind of Asphalt	Original A.C. Pen. at 77° F.	Original A.C. Duct. at 77° F.	Ottawa Sand Mix (A) 1 Hour at 350° F. Pen. 77° F.	Ottawa Sand Mix (A) 1 Hour at 350° F. Ductility 77° F.	Ottawa Sand Mix (A) 1 Hour at 350° F. Ductility 60° F.	Sheet Asph. Mix (B) 1 Hour at 350° F. Pen. 77° F.	Sheet Asph. Mix (B) 1 Hour at 350° F. Ductility 77° F.	Sheet Asph. Mix (B) 1 Hour at 350° F. Ductility 60° F.
1	California	55	110+	34	110+	110+	28	110+	14.5
2	*Mexican	54	110+	34	17	7	127	11	0
3	Venezuelan	55	110+	26	6.5	4.5	20	6	4
4	Columbian	51	110+	23	11	4	—	—	—
		64	110+	—	—	—	22	4.5	3.5
5	*Mexican	54	110+	20	5	4	23	6	4
6	Venezuelan	54	110+	24	8.5	4.5	24	7.5	4.5
7	Domestic, Cayuga	55	110+	25	10	4	—	—	—
		63	110+	—	—	—	25	7.5	3
8	*Mexican	55	110+	24	9.5	5	20	5.5	4
9	Domestic (Law)	58	110+	31	22.5	5.5	—	—	—
	Texas	64	110+	—	—	—	25	5	3.5
10	Domestic, Arkansas	51	110+	29	15.5	5	—	—	—
		63	110+	—	—	—	24	4	3
11	*Mexican	56	110+	22	5	3.5	—	—	—
		65	110+	—	—	—	25	3.5	3
12	Domestic (Whiting)	56	110+	36	8	8	31	4.5	3

* The four samples of Mexican Asphalt are from four refiners.

Table 4 includes data obtained by these tests using various asphalts. The similarity of results between the two tests, and with results of the tests shown in Tables 1, 2 and 3, is striking. Since these tests were made, Mr. Shattuck has designed a miniature mixer in which samples can be mixed under accurately controlled conditions of air exposure and temperature, and the results of recent tests are even more consistent.

Tables 5, 8, 9 and 10 are based on tests made under the writer's direction. Table 6 is from tests made by Mr. F. T. Maddocks in 1924 in the Laboratory of the California Division of Highways. Table 7 is from a report on "Investigation of Behavior of Asphaltic Cement Admixed with Sulphates" made to the California Division of Highways by the Industrial Testing Laboratory of Los Angeles in 1923, and not believed to have been previously published.

Aging in Pavement is Due to Air Oxidation

A study of the data in Tables 1 and 2 on laboratory mixtures, compared with results of tests shown in Table 4 on pavement samples, indicates that the reductions in penetration and ductility are due to oxidation in the presence of air. This is particularly confirmed by the work of Raschig and Doyle[10] in the table on page 2 of their paper where they show comparative results obtained by air blowing asphalt and by heating sheet asphalt mixes and then extracting the asphalt. It is also confirmed by the work of Nicholson[9] whose results on air blowing tests are very

similar to those generally obtained by heat treatment of asphaltic mixtures. The tests reported herein, and in the many papers referred to, definitely establish that the changes brought about in asphalt through aging, are the result of removal of hydrogen from the asphalt by air, or by sulphur, or both, and also that properties similar to those found to exist in a pavement can be produced immediately by accelerated oxidation, utilizing heat treatment and aeration.

The way is therefore open to measure changes wrought by time and weathering in asphaltic binders in existing pavements, and by accelerated tests to forecast with reasonable accuracy what will happen.

CHANGES IN ASPHALTIC MATERIAL DUE TO OXIDATION

Penetration

The consistency of an asphaltic material is measured by the penetration test in paving asphalt and by viscosity and float tests in softer binders. It is here asserted that the penetration test, alone, is the least important of all physical tests of asphalt and that narrow specification limits in penetration on any grade of asphalt are purposeless. With wide allowable specification limits such as 50-100, 100-200, 200-300 and 300-350+, the test is valuable for classifying asphalts into various grades by relative consistency. The theory held by some engineers that hard asphalts are necessary to produce stable pavements, is unsound and has led to the use of hard asphalt, still further hardened by heating during paving operations.

Skidmore and Abson[6] point out that they have long warned against the use of hard asphalts. Hubbard and Gollomb[7] urge the use of the softest asphalt which will give stability. The writer has been in responsible charge of the manufacture of at least half of all of the emulsified asphalt heretofore used in the United States. He has never permitted emulsified asphalt to be manufactured for road surfaces, or street pavement, from asphalt having a penetration of less than 150, and has always recommended 250-350 penetration asphalt for emulsification where it is to be used in thin pavements, on flexible bases or with silty or absorbent filler. In no instance has instability resulted from softness of binder, because the use of excess asphalt is always avoided.

In many of the research papers the hardening of asphalt is measured and reported solely as decrease in penetration. Decreased penetration is significant only because it usually indicates some change in other properties. A decrease in penetration usually accompanies a decrease in ductility, but not in any direct proportion. In Table 1, for example, with California asphalt and 120-minute heating, we find a loss of over 60% in penetration, but no measurable loss in ductility. The Texas asphalt here tested has a loss of 55% of its penetration and 96% of its ductility. The same lack of relationship between penetration and ductility is also found in Tables 2, 4, 5, 6, 8, 10 and even in 7, where oxidation is accelerated by the admixture of sulphurous compounds with the asphalt.

Penetration, at most, could only be indicative of hardness in the pavement. If stability were dependent on hard asphalt, which it is not, then increased penetration, alone, should indicate increased stability, a thing to be desired.

TABLE 5

EFFECT OF HEAT TREATMENT ON VARIOUS ASPHALTS

Tests on Asphalt as Received and After Extraction from Heated Mix

	East Texas	East Texas	SM Calif.	SM Calif.	Mid-way Calif.	Mex. (Panuco)	Ore-gon B Wyo.	Gar-land Wyo.
Penetration at 77° F.								
As Received	93	176	67	190	173	240	127	229
From Heated Mix*	33	45	19	45	86	66	52	53
Penetration Loss %	64	74	72	76	50	73	59	73
Ductility at 77° F.								
As Received	150+	139	150+	124	150+	150+	150+	126
From Heated Mix	6	7	12	61	150+	81	150+	101
Ductility Loss %	97	95	92	51	0	46	0	20
M. P.								
As Received	122	106	121	104	100	107	.....	101
From Heated Mix	158	155	159	138	114	.....	134	126
Sol. 86° Naphtha								
As Received	73.0	75.4	61.9	76.6	90.4	74.9	.....	81.2
From Heated Mix	66.5	68.8	.....	.....	84.2	67.6	77.2	72.6
Sol. CCl_4								
As Received	99.8	99.7	99.7	99.9	99.9	99.9	.....	99.9
From Heated Mix	99.7	99.7	.....	.....	99.9	99.8	99.8	99.8
Sulphur %	4.46	4.67	5.93	5.92	1.39	5.84	4.25	4.39
Ash								
As Received	.....	.....	.....	.....	.....	.10	.....	.....
From Heated Mix	.....	.....	.08	.12	.10	.16	.30	.30
Fixed Carbon								
As Received	17.2	15.5	11.0	10.2	7.2	12.4	.....	14.4
From Heated Mix	.....	.....	12.5	10.8	9.4	13.5	16.2	12.8
Wax %	2.09	2.12	0.40	0.51	0.20	0.39	.....	1.47
Sp. Gr.	1.023	1.017	1.041	.....	1.0062	.....	.....	1.023
Oliensis								
As Received	Neg.	Neg.	Neg.	Neg.	Neg.	Neg.	Neg.	—
From Heated Mix	Pos.	Pos.	Pos.	Pos.	Neg.	Neg.	Neg.	Pos.
Loss at 325° F. %								
As Received	0	0	0.6	1.3	0	.....	.....	0.2
From Heated Mix	0	0.1	0.4	0.6	0.2	.....	0.1	.....
Penetration								
After 325° F. As Rec.	71	145	51	107	139	196	.....	180
From Heated Mix	31	43	17	39	78	.....	46	49

***Hot Mix:** 93% of 10/20 mesh sand and 7% asphalt heated to 350° F. Immediately after mixing, placed in insulated oven heated to 350° F., heat turned off and oven allowed to cool. Temperature dropped to 200° F. in 5 hours and to atmospheric temperature (70° F. +) in 15 hours. Asphalt extracted by Abson method.

Ductility

Ductility, in an asphaltic binder, is of paramount importance. It is reasonable to assume that an asphalt which is highly ductile, and which maintains good ductility after weathering, will produce a tough, flexible pavement free from cracks, other conditions being in order.

In Table 2, it is indeed startling to find an asphalt reduced from 100+ductility at 77° to 2 cm., (98+% decrease) in 120 minutes. This is more than the reduction found in Table 3 in a pavement 26 years old,

and much worse than the average ductility found in 16 badly cracked streets only 6 years old (last item in Table 3). In Table 5 heating to 350° F. following by slow cooling to atmospheric temperature in 15 hours is found, with one asphalt, to reduce ductility from 150 cm. to 6 cm. and with several other asphalts, almost equally serious loss in ductility is noted. Three asphalts in Table 5, after this test, still retain ductilities over 100 cm.

The writer believes that some type of ductility or elongation test should be made at other temperatures than 77° F., but also believes that the present ductility test at low temperatures, 39.2° F.; 32° F.; or 0° F., has no parallel in service conditions and, therefore, lacks real significance. The test at 77° F. is made at 5 cm. (2 inches) per minute and at 32° F., usually 1 cm. per minute. No pavement could possibly contract at the rate of 5 cm. (2 inches) per minute. At 60° F., or 77° F., it is reasonable to expect that asphalt should be very ductile and high speed tests at these temperatures with high ductility results probably indicate good adhesive and cohesive properties. The belief is here expressed that ductilities at low temperatures should be tested at very slow speeds, say 1 mm. per minute, and even this speed is more rapid than any possible shrinkage in a pavement.

A new test should be devised which could be used for measuring ductility of mixtures at all temperatures, simulating as nearly as possible, service conditions.

It is suggested that an elongation test be made on specimens molded from laboratory mixes, or remolded from road samples. The specimens might be shaped something like cement mortar tensile strength briquettes, but should have a much larger cross-section and probably be considerably longer between the clips in the machine, because of the relatively low tensile strengths and to permit of measurable elongation before failure. It is suggested that the load be applied at a uniform rate and very slowly, and that elongation be recorded as the load is increased.

If such a test could be made on good and bad pavements of various ages, and by comparative tests related to air blowing and hot mix extractions ,it might be standardized to a point where it could be substituted for present extraction tests which are tedious. It would also eliminate uncertainty if it could be so conducted as to avoid heating of old mixes, because some of the hardening in asphalt is reversible and disappears when the asphalt is re-heated. This is brought out in a paper on the effect of sulphur on asphalt, by Bencowitz and Rowe[14] and is noted in many of the papers referred to. Pavements in service are not re-heated and should be tested, if possible, without, to any extent, restoring the ductility by heat treatment in testing.

Asphaltenes

The asphaltene content of asphalt (the portion insoluble in 86° Naphtha), increases with air oxidation. In Table 5 the decrease in solubility of the asphalt in 86° Naphtha (increase in asphaltenes) in asphalts recovered from heated mixes, is shown for most of the asphalts, and ranges from 7 to 9%. The increase is approximately uniform for all of the asphalts, and is not widely variable as are the decreases in penetration and ductility. The increase in asphaltene content is probably not

significant, except as it indicates that chemical changes actually occur due to oxidation. The asphaltene content of asphalt is believed by some engineers to represent the portion of asphalt which is inert and inoperative as binder. It has been definitely known that lesser amounts of asphalt are required in a mix with some asphalts having low asphaltene content than are required with asphalts having a high asphaltene content. On this basis an increase in asphaltenes by oxidation would indicate a reduction in the amount of effective binder. With a high natural asphaltene content and a further increase as a result of aging, the amount of effective binder might be reduced to a dangerous point.

Effect of Sulphur in the Aging (Oxidation) of Asphalt

It has long been known that sulphur and its compounds are effective as hardening agents for asphalts. Patents were obtained by Day in 1875 and by Dubbs in 1892, for adding sulphur to asphalt and then heating until the evolution of gas ceased. According to Abraham[15] the asphalt so produced was similar to air-blown asphalt, except that it lacks ductility. The hardening action is thus explained by Abraham:

> "Under the influence of heat, sulphur has the same condensing effect on bituminous substances as oxygen, since it results in the removal of hydrogen, but in this instance in the form of gaseous sulfide. The reaction may be roughly represented as follows:
>
> $$Cx\ Hy + S = Cx\ Hy + H_2S$$
>
> "A certain amount of polymerization also takes place."

TABLE 6

EFFECT OF SULPHUR ON PENETRATION AND DUCTILITY OF ASPHALT

50 Gram Sample **Original Penetration Asphalt 58**

Tests After Heating at 250° F.—2 Hrs.	Blank	Admixture				
		2.66%	Sulphur			
		$CuSO_4$*	0.50%	1.00%	1.66%	2.25%
Penetration 77° F.	52	29	44	39	33	27
Ductility 77° F.	200	110	110	110	103	92

*Based on weight asphalt, 2.66% copper sulphate contains 0.58% sulphur.

TABLE 7

EFFECT OF SULPHUR COMPOUNDS ON DUCTILITY AND PENETRATION OF ASPHALT

Admixture	% Admixture	Sulphur Content Admix. %	Pen. at 77° F.	Ductility at 77° F.
None	As Received		47	150
Copper Sulphate $CuSO_4 .5H_2O$	2.5	.321	17	85
Aluminum Sulphate $Al_2 (SO_4)_3 .18H_2O$	2.5	.362	44	52
Magnesium Sulphate $MgSO_4 .7H_2O$	2.5	.325	44	126
Ferrous Sulphate $FeSO_4 .7H_2O$	2.5	.288	46	150
Zinc Sulphate $ZnSO_4 .7H_2O$	2.5	.277	45	150
Lead Sulphate $PbSO_4$	2.5	.264	29	150

Interesting tests showing the effect of heating asphalt with a sulphur admixture have been presented by Brooks and Humphrey[16].

A few years ago a proprietary pavement was marketed in the United States in which Copper Sulphate was added to asphalt prior to mixing. Cracks up to 3 inches in width quickly appeared in such pavements and the generally disastrous results are still fresh in the memory of paving engineers.

Probably all petroleum asphalt contains some sulphur. Table 5 shows the sulphur content of seven asphalts from six sources. The asphalts which show the greatest loss in ductility and penetration have high sulphur content. The three asphalts with the lowest sulphur content show the least loss of ductility. Two of the asphalts having a sulphur content of over 4% still show good ductilities after heating. As a probable explanation, the thought is offered that sulphur exists in many combinations and that the effectiveness of the sulphur varies with the different compounds in which it occurs. This is illustrated in Tables 6 and 7. Table 6 shows the relative effect on ductility and penetration of pure sulphur and of copper sulphate when each was admixed with asphalt and held at 250° F. for two hours. 2.66% of copper sulphate reduced the penetration of the asphalt from 58 to 29, or 50%. A blank sample in the same 2 hours of heating lost only 6 points in penetration. The ductility with copper sulphate dropped from the 200 cms. ductility in the blank to 110 cms. (2.66% of copper sulphate contains 0.58% of sulphur). Pure sulphur was added in the same manner as copper sulphate in varying quantities, and it will be noted that it took approximately four times (2.25% to 0.58%) as much pure sulphur as it did when the sulphur was in the copper sulphate compound, to produce the same reduction in penetration and ductility. Table 7 shows a number of sulphurous compounds added to asphalt. All reduced the penetration, but several did not reduce the ductility. This table shows the varying effectiveness of sulphur compounds has no relation to actual sulphur

content. It is undoubtedly true that sulphur is present in different forms in different asphalts, and its effectiveness would therefore vary widely, as is shown to be the case in Table 5. In Table 5, the greatest drop (97%) in ductility is found with 4.46% of sulphur present, while the Mexican (Panuco) Asphalt has about half this loss in ductility (46%) yet has a sulphur content of 5.84.

If sulphur is largely responsible for the loss in ductility, this would indicate that the sulphur compounds in the East Texas asphalt (Cols. 1 and 2) are somewhat more active than those in Mexican. The sulphur content in the California Midway is very low and shows no effect on ductility and the least on penetration.

Oxidation of asphalt could, of course, occur in the absence of sulphur, but it seems to be conclusively established that sulphur can greatly accelerate the early hardening of asphalt and will do so unless the sulphur content is low or the sulphur in a form in which it is not active.

Wax in Asphaltic Materials

The wax content was determined on all but one of the samples reported in Table 5. The two asphalts which showed greatest loss in ductility are reported in the first two columns, and these happen to be the ones with the highest wax content. The best asphalt, as measured by ductility tests in this table, shows a low wax content, but the asphalts in Column 3 and 4 with low wax content also show very serious losses in ductility, therefore it cannot be said that wax content within the limits of the samples had any influence on ductilities.

Benson's[11] comments on the results of his research confirm the generally accepted theory that wax tends to produce poor adhesive properties in asphaltic materials, but there is no evidence that it hastens oxidation.

Oily Constituents and Their Effect on Oxidation Products

It has been generally shown that the aged asphaltic binder in a pavement closely resembles air oxidized asphalt obtained by the air blowing of heavy residuals.

TABLE 8

EFFECT OF AIR BLOWING ON RESIDUAL OILS OF DIFFERENT GRAVITIES

All residual oils previously steam-refined from identical crude to gravities indicated.

A. P. I. Gravity of Residual Oil Before Air-Blowing	Tests on Resulting Air-Blown Asphalt				Air-Blown Asphalt Recovered from Hot Mix*		
	Pen. at 77°F.	R. & B. Melting Point	Ductility at 60°F.	Ductility at 77°F.	Pen. at 77°F.	R. & B. Melting Point	Ductility at 60°F.
11.0 Gravity	58	132.5	32	72	37	147	6.0
10.5 "	56	130.0	112	150+	40	139	8.0
10.0 "	60	125.0	44	150+	42	135	10.5
9.5 "	59	124.0	120	150+	41	135	12.0
9.0 "	56	122.0	150+	150+	40	130	34.0
8.5 "	60	119.5	150+	150+	39	130	139.0

*Mix made with 20-30 mesh Ottawa sand with 7.7% asphalt. Mechanically mixed in open-air 5 minutes at 370° F., then extracted by Abson method.

Low penetrations are always found in asphaltic material extracted from old pavement. It, therefore, seems reasonable to assume that, durability being considered, the most suitable asphaltic binder would be expected to be that resembling a charging stock, which, in refinery air blowing would give the most ductile asphalt of similar penetration.

A mixture of asphalt and gas oil or lighter solvents would not be expected to produce a ductile air (blown) oxidized asphalt. Heavy oils in a heavy residuum are converted to asphalt by blowing. A suitable heavy oil will yield as much as 95% of air-blown asphalt. The relation of gravity of oil to ductility of the oxidized asphalt produced therefrom is shown in Table 8. All of the residual oils in this table are from the same crude. It will be seen in this table that a residual oil of 8.5, A. P. I. gravity at 60° F., produced an asphalt by air blowing with a ductility of 150+ cms. at 60° F., while an 11 gravity residue produced an asphalt of the same penetration, but with only 32 cms. ductility at 60° F. When these asphalts were mixed with hot sand and recovered, the differences in ductility were even more startling. At 60° F. they range from 139 cms. for asphalt made from 8.5 gravity residual oil to 6 cms. for asphalt similarly produced from the lighter 11 gravity oil. An 8.5 A. P. I. gravity residual from this crude is approximately a 200 penetration asphalt and an 11 gravity residual is approximately an SC-5. In other words, air oxidized asphalt, to have high ductility at 60° after recovery from a hot mix, must have been blown from residual much heavier than SC-5, and even heavier than an SC-6, which, with the same crude has an A. P. I. gravity of about 9.1.

Straight run steam refined SC oils contain heavy asphaltic residues fluxed with medium heavy oils. A blend of heavy asphaltic residues and light oils would not be expected to produce a ductile asphalt by air oxidation and it is difficult to see how it could be expected to produce ductile asphalt in a road after it is oxidized. An MC cutback is such a blend of asphalt and light oils.

Possibly this may throw some light on Benson's[17] observations to the effect that asphalts recovered from surfaces constructed with MC cutbacks were found to have lost most of their ductility in less than one year. The heaviest MC cutback, MC-5, has an A. P. I. gravity of about 11 when made from the crude used in producing the heavy residuals in Table 8. This is about the same as the gravity of the residual oil which gave the poorest results in Tabel 8. MC-1 to MC-4 are still lighter and the results should be even less favorable as far as ductility is concerned.

It is suggested that those interested in verifying the effect of oxidation on road oils or cutbacks can readily do so by taking samples from existing pavements, as Benson did, and recovering the asphalt by the Abson Method and also by making laboratory mixes and heating until the asphaltic residue obtained by the Abson Method has a penetration between 40 and 60, then noting the ductility at 60° F. or at 77° F. The Nicholson[9] Method of blowing an asphalt or road oil could also be used with modifications. It would probably be desirable to start blowing at a temperature of 150° F. and raise the temperature as oxidation proceeded, until a final temperature of about 450° F. is reached and the residue brought to the desired penetration.

The same studies should be applied to RC cutbacks and to mixes made with them. Low ductilities obtained in these tests should constitute a danger signal to a careful engineer.

At this point it seems well to comment briefly on the relative merits of the Abson and Bussow methods of extraction. Steinbaugh and Brown[8] made an extensive study of both methods. The data which they have published seems to justify a preference for the Abson method. This confirms our own observations, and the Abson method has been most generally used by investigators. Steinbaugh and Brown[8] show a drop in ductility averaging about 12 cms. in the process of extracting and recovering an asphalt by the Abson method. This can be taken into consideration in interpreting data and is not seriously objectionable.

Effect of Aging on Emulsified Asphalt Coatings

In the proper manufacture of emulsified asphalt for road surfaces, two grades of asphalt are used: the asphalt in one grade, prior to emulsification, has a penetration of 150 to 250; the other, 250 to 350. Asphalt is not (or should not) be heated above 250 or 275° F. during the manufacturing and is applied at, or slightly above, air temperatures. In this way, the oxidation losses at time of use are avoided. After application, of course, air oxidation begins. It should be noted here that the 150 to 250 penetration asphalt corresponds closely to the 8.5 A. P. I. gravity residual oil which gave most satisfactory results in air blowing in Table 8. The asphalt left on the stone in the road in aging should, if the theories advanced are sound, take on, in aging, the properties of an air-blown asphalt from the same A. P. I. gravity residuum, and it does.

TABLE 9

EFFECT OF AGE ON PHYSICAL PROPERTIES OF ASPHALT IN EMULSIFIED ASPHALT PENETRATION PAVEMENTS

		Asphalt as Used					Asphalt Recovered From Pavement			
Date Built	Name of Project	Pen. at 77° F.	Duct. at 77° F.	M.P. R & B	Sol. 86° Naph-tha	Age at Test	Pen. at 77°	Duct. at 77°	M.P. R & B	Sol. 86° Naph tha
1927	San Ardo, Calif. State Highway	175	150+	105	91.2	8 Yrs. 6 Mos.	56	150+	118	83.1
1931	10th St., Oakland, Calif.	180	130	101°F.	91	4 Yrs. 2 Mos.	61	150+	117	73.7
1931	10th St., Oakland, Calif.	180	130	101°F.	91	6 Yrs.	48	150+	121	Not Tested

Table 9 shows the results of extractions by Abson Method from emulsified asphalt penetration pavements 4 to 8½ years old. It will be found by comparison of Tables 8 and 9 that the residual asphalt in the road samples does closely resemble that of the asphalt produced by air blowing the product designated as 8.5 gravity residual oil which, as stated, is an asphalt of about 200 penetration. Ductilities are high with the artificially blown asphalt and also with the asphalt recovered from the pavements. The drop in penetrations is similar in both tables. The melting point has increased in both cases, although somewhat less in the old pavement samples than in the air-blown samples reported in Table 8. The asphaltene content has similarly increased in both cases.

The asphalt used for emulsification was the same as that used for the tests reported in Table 8, and is the same "Midway California" asphalt shown in Table 5. The results there shown (Table 5), as to ductility, penetration and asphaltenes, after the heat test, are substantially the same as those in Table 9 obtained from field samples.

Table 10 shows results of tests made on cold laid pavement after 18 months service in a region with hot summers and snowy winters. Tests are also shown made on the mix as laid. In producing this pavement part of the binder was in the form of hot asphalt of 60 penetration added to aggregate at 200 to 240° F. The mix was then immediately cooled to about 180° F. by the addition of emulsion, which constituted about ⅓ of the binder. The bitumen in the emulsion was soft enough to flux the 60° asphalt and to produce a penetration in the extracted binder of 193. A comparison of the results in this table with those in Tables 8 and 9 again shows that the residual asphalt after 18 months service has all of the ductility characteristics of an air-blown product. The penetration had changed relatively little.

TABLE 10

AIR OXIDATION IN COLD LAID PAVEMENT

Holbrook to Lupton, Arizona — Completed November, 1934
Asphalt by Absorption Extraction

Physical Tests of Extracted Binder	Original Mix*	Road Samples May, 1936
Penetration at 77° F.	193	183
Penetration at 32° F.	32	33
Ductility at 77° F.	150+	119
Ductility at 32° F.	101	29
Solubility 86° Naphtha	92.0	81.5

*This mix made by heating aggregate (dense graded mix) to 200-240° F. and adding approximately 6.5% of binder. The binder consisted of 2⅝% emulsified asphalt tempering fluid added to aggregate cold, followed by 4⅛% of 50/60 penetration asphalt heated to 300° F. Mixture 40 to 80° F. when laid.

The asphaltic materials used in this last job (Table 10) were not from the same crudes as those emulsified for the jobs shown in Table 9, but are very similar.

It is repeated here that the quality of a properly manufactured asphalt emulsion depends primarily on the quality of the asphalt from which it is made. Tables 9 to 10 indicate that good asphalt properly emulsified shows no serious loss in useful properties after long service.

It is recommended that samples of asphalt before emulsification always be tested as received and again after oxidation to determine whether a durable asphalt is being used.

CONCLUSIONS

1. Asphaltic binders for use either hot or cold, which lose ductility rapidly in oxidation tests, will similarly oxidize and lose ductility in pavements, with resultant brittleness and cracking.

2. The heat treatment or air blowing methods of testing can be applied to liquid asphaltic binders prior to use, as well as to paving asphalt, and can be expected to show whether a durable ductile binder will be produced in the pavement when it has aged.

3. Sulphur probably contributes to early hardening and loss of ductility in asphaltic binders.

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Sand Asphalt Mixtures in the Southeastern States and Factors Influencing Selection of Bituminous Binders

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The state highway systems of the southeastern states, particularly Alabama, Georgia, Florida, North Carolina and South Carolina contain a large mileage of all weather surfaced roads which consist of a thin bituminous wearing course about ¾" thick placed on a base of compacted soil. In fact this type of road comprises the majority of roads on highway systems in these states as it has proved economical and very satisfactory in areas where traffic densities are low. By careful attention to design and construction details, this type will satisfactorily serve where traffic densities are considerably above the average for these states.

The soils for these bituminous surfaced roads must be very carefully selected. The clay content must be relatively low and the clay must not be too plastic. In addition the sand portion must be well graded or the soil will lose stability when the moisture content increases and it will then be unable to carry the traffic loads. These soils can be selected by laboratory tests based on experience over a long period of time and they are generally well distributed over the states but in certain areas suitable soils do not exist and methods of stabilizing the materials which are encountered have been studied in an effort to extend the building of low cost roads to these areas. A low cost road in these locations is usually very desirable because the area is sparsely populated and the traffic does not justify the use of high type pavements. Several methods of stabilizing soils are being used. For clay soils, cement and tar have been used satisfactorily but neither of these methods is, as yet, perfected. Sandy soils and those containing a small quantity of clay can be stabilized by the addition of a bituminous binder and a description of the methods used for stabilizing these soils will be presented here.

A much greater mileage of asphalt stabilized base or "Sand Asphalt," as it is commonly known in these states, is in service than either of the

other types of stabilization mentioned above and it is the only method o soil stabilization used in this part of the country which is considered t have progressed beyond the experimental stage. By using designs an methods of construction which have been developed over a period o years, a sand asphalt base can be constructed with full assurance tha the results will be satisfactory.

The states mentioned herein all contain large areas in which th soil is almost pure sand or sand which contains only a little clay. I general, these areas are near the coast although sand dunes and sand areas are found far inland due to the geological formation of the country

The author would like to give due credit to the man who started "Sand Asphalt" if his name were known but it is very difficult t ascertain even which of the highway departments first began experiment in this type of construction. North Carolina used local sand in a hot mix method fourteen years ago, but the resulting road was hardly a low cost type as we now define it. Florida began work with cold mixes 8 years ago and South Carolina started in 1925. In Georgia and Alabama the use of this method is more recent. At the present time Alabama has approximately 23 miles of this type, Georgia 150 miles, Florida 485 miles, North Carolina 55 miles and South Carolina 275 miles.

A brief description of the method of construction is as follows:

The roadway is first graded and brought to the correct cross-section, and, if the subgrade soil is unsuitable sandy soil is brought in from borrow pits. The sand is mixed cold with a liquid bituminous material, usually rapid curing type cut-back asphalt, having a viscosity sufficiently low to permit application by slight warming and mixing without the addition of heat. A sufficient amount of materials must be included in this mixture so that the thickness and width will be satisfactory after compaction. The mix must be cured until most of the volatile portion of the bituminous material and the moisture evaporate and it is then spread to the correct cross-section and compacted. It is necessary to maintain the surface by blading during the compaction period in order to obtain a satisfactory riding surface. Usually a wearing surface of some type is applied to the base. In some instances the wearing course consists of a hot application of asphalt covered with slag or stone chips applied directly on the base and in some locations the top 1″ or 1½″ of the original mix is enriched by the addition of more bituminous material and this when compacted serves as the wearing course.

The selection of this type of construction for any project is dependent on the availability of sandy material. Projects of this kind are usually confined to areas where the soil in the roadway after grading is completed is suitable for inclusion in the base. However, in areas where the subgrade cannot be used but sandy soil is available within a reasonable haul distance, this type of construction can be used economically if sand clay or top soil suitable as base material for bituminous surfacing is not available.

Rapid-curing cut-back asphalts are used almost exclusively as the bituminous material in this type of construction, but experiments using

tars, emulsions, slow and medium curing liquid asphalts and Colprovia materials consisting of flux oil and powdered asphalt have been tried in several locations. The use of RC type cut-back asphalt is probably the result of the satisfactory behavior of this material in the early experiments, and now the contractors and engineers are familiar with the way in which this material acts and consequently know from experience what manipulation and operations are necessary in order to secure satisfactory results. The limited experiments with tar have been satisfactory and it is believed that this material could be used as an alternate to RC type cut-back asphalt except for the fact that tar in this section of the country is more expensive than cut-back asphalt and is usually eliminated from an economic standpoint. The asphalt emulsions have given considerable trouble during mixing if the aggregate is dry due to the fact that emulsion breaks before it can be distributed and the asphalt forms into balls which are very difficult to break up and consequently it is extremely hard to get a uniform coating of asphalt on all of the particles of sand. When the aggregate is wetted so as to avoid this trouble, the emulsion does not "break" or cure much below the surface and this mixture does not have the stability or water resisting properties which are necessary. The slow-curing liquid asphalts which are available in this area consist of a straight run material which does not possess much binder, although tests show that it contains 60% asphalt of 100 penetration. It has been used successfully as a dust layer on stable soils and unsuccessful attempts have been made to stabilize rather high clay soils with this material.

Medium-curing liquid asphalts have been used in only a few experiments. It has been used with some success as a primer but where it has been tried in mixes, trouble has resulted from cracking after compaction due to an excess of volatile matter in the mix. The operators who were accustomed to working with RC cut-backs were under the impression that a sufficient quantity of volatile material had evaporated before compaction but they did not recognize that evaporation of the heavier kerosene solvent used in this type of cut-back is much slower than the naphtha used in RC cut-backs.

RC type materials can be adopted to both plant and road mixes by varying the viscosity. It appears that the naphtha which is used as a solvent in this material evaporates more completely than kerosene solvent and, therefore, does not have the same tendency to form a greasy mix. During cold weather, mixtures with RC cut-backs have been compacted without waiting for the curing period if the moisture content is sufficiently low.

Methods for determining the quantity of bitumen required to stabilize sandy soils vary in each of the states. It is, of course, necessary to use sufficient asphalt to bind the particles together. In general none of the known formulae have proved satisfactory and the quantity of asphalt is based on some laboratory tests—usually a mechanical analysis interpreted in the light of experience over a period of years. Trial mixes are used in North Carolina to a large extent. In South Carolina the quantity is determined by comparing the mechanical analysis of the soil under consideration with that on a project previously constructed with soil of similar grading and the bitumen content is set at a point known by experience to be satisfactory.

In Florida, the sand is tested for stability in a modified Bearin Value Machine. The use of this machine has eliminated trouble whic was experienced in Florida with sand asphalt mixes which would no "set up." It was discovered that the sand in these mixtures was ver uniform in size, possessed no cohesion and was easily displaced due t the roundness and uniform size of the grains. The condition was correcte by the addition of other aggregates to the mix. Sands which have bearing value of 25 when tested in this machine have been found to giv satisfactory stability on the road. The other states have not experience the same trouble with lack of stability of the aggregate but they hav found that the quantity of bitumen is more critical for sands of coars uniform grading and that shoving and rutting is likely to occur if th bitumen content is too high. However, if the bitumen is proportione correctly there has been no trouble from instability of the finished base

The range of materials used has varied from coarse sand having a sur face area of 36 square feet per 100 pounds to fine sand clay having a surface area of 150 square feet per 100 pounds. The bitumen contents vary from 3 to 8% depending chiefly on the grading of the sand and on the quantity and type of clay or other foreign matter such as marl. In genera the bitumen content is set at the minimum quantity which will provide sufficient binder to hold the soil together. This results generally in a mix which appears to be lean and the base will dust and ravel slightly under traffic before the wearing course is applied. Of course, bases containing the minimum quantity of bitumen for stabilization are more economical than richer mixes and experience over a number of years shows that the durability is satisfactory. Apparently nothing is gained by the addition of sufficient bitumen to eliminate all raveling or dusting.

When soil having as much as 15 to 20% clay is used in the mix, the appearance of the finished mix is brown rather than black as sand mixes are. Even when the bitumen content is increased, it is still very difficult to get the two types of soils to give a mixture of the same color. However, the mixes with this quantity of clay soil are more stable than the sand mixes and results show that a small quantity of clay or fine material is very beneficial in increasing stability. The Hubbard-Field Stability machine has been used in a limited way in testing these mixes and it has been found that the stability on a mixture with sand may be as low as 600 pounds per square inch whereas the stability of the mix with sand clay soil may be as high as 2,000 pounds per square inch. Differences such as these have made it impossible to use this machine as a criterion in setting bitumen contents. Two experiments in South Carolina in which RC type cut-back asphalt has been used in heavy clay soils have proved unsatisfactory. In these experiments, the cut-back asphalt was mixed in the soil and the mixture was compacted at optimum moisture content. The mix cracked and softened during wet weather, showing that the compacted mixture was not waterproof.

Mixing is performed either by a traveling mixing plant, a stationary mixing plant or the road mix method. The early experiments and practically all of the work up to 1937 was constructed by the road mix method. It has proved very satisfactory and if mixing is carefully performed a homogeneous and uniform mix can be obtained. It has a number of dis-

lvantages which are inaccurate control of depth, inaccurate proportioning asphalt, non-uniform mixing and delays and damage from bad weather. hese objections added to the advantages of mixing machines are gradually iminating the mixed-in-place method from general use. Several types of avelling mixing machines have been used on this work and all of them pparently do good work. The asphalt is better distributed for machine ixes containing the same quantity of asphalt are richer in appearance an road mixes. It is sometimes possible to reduce the quantity of cut-ack asphalt as much as 15% for a 5" base when machines are used. his reduces the theoretical asphalt content by almost 1% by weight. elays due to adverse weather are reduced because the material can be etter protected. The actual proportioning of the mixture is generally ependent on the size and moisture content of the windrow ahead of he machine and, therefore, the windrow must be carefully formed if ac-urate results are obtained. There is still room for improvement in the esign of the travelling mixing plants as none of the present machines ppear to possess all of the advantages which might be included. A central ixing plant for small jobs particularly shoulder work is being widely sed in South Carolina. This type of plant possesses all of the advantages f the traveling mixing plant and when used for small projects, it is ore economical than road mix methods.

The percentage of moisture in the soil at the time of mixing is very nportant. A small quantity of moisture approximately 6 to 8% in the verage graded sand is of great benefit in mixing but an excess of water o that the moisture in the sand is considerably above the saturation oint or there is free water present is deterimental. When over 8% mois-ure is present, mixing is much more difficult and drying after mixing extremely expensive. Also, it appears that the additional manipula-on required for drying causes some of the asphalt to disappear or become nactive as binder. An explanation for this particular trouble has not been ound. When free water is present the sand grains are hard to coat nd even when they appear to be coated the asphalt can be displaced rom them by additional mixing. Wetting of the mixture immediately fter the asphalt has been applied is also very troublesome. Water appears o wash the asphalt from the sand grains and the asphalt forms into mall balls which are very difficult to redistribute in the mix. It is usually ecessary to add additional asphalt to a mix which is wetted before any f the naphtha evaporates. None of the stripping tests which are com-nonly used have been applied with success in predetermining the affinity f sands for asphalt. The particles are essentially quartz and most of hem are rounded.

Most of the volatile materials in the mix must be evaporated before he mix is spread and compacted and experience is of great benefit in letermining when the mix is in condition for compaction. It must still ontain a sufficient amount of volatile material so that it can be spread niformly but the moisture content and naphtha content must be relatively ow or the mix will crack and ravel after compaction. Laboratory tests vill not measure definitely when the mixture is in a condition satis-actory for compaction because of the great variations in the soils and he opinion of an experienced man in watching the material as it comes

from the road machine blade or in testing it by feel is at present mo definite. The quantity of moisture is much more critical than the naphth Experience in South Carolina shows that the mix should not conta over 6% moisture when it is compacted.

Mixes in which the soil is high in silt or clay content, up to 30 passing the No. 200 sieve, can be compacted with considerably mo naphtha or moisture than those in which fine particles are absent. has been mentioned before that fine particles increased the stability, b they also decrease the curing period and it has been possible to compa a few experimental sections in which the soil contained a high percer age of fine particles almost immediately after mixing.

Compaction is generally secured by traffic over a period of sever weeks. but in Florida a 3 to 10 ton tandem roller is used and the co paction period is consequently shortened. Loaded trucks can also be us if it is desirable to secure compaction quickly. Compaction by traffic h the objection of forming a crust on the surface and there is usually considerable difference in density of the base in the top and bottom po tions. However, over a long period of time traffic appears to consolida almost the entire base. During the compaction period the surface mu be machined very frequently in order to obtain an acceptable riding su face. In Florida, a road machine with the blade heated at the back b oil burners has proved effective in drying the soil and in the final finis ing. The added heat increased the workability of the mix and evaporatic but it is most useful in drying the raw soil before the addition of t bituminous material. This equipment has been tried in South Carolina ar it was observed that the heat damaged the mold-board of the machine such an extent that it would bend when pulled against a stump or simil object. With special blades made to withstand the heating, it is believe that this device would aid very materially in construction work, especiall in the cooler months.

A sheeps foot roller has been used in some instances for compactic but this equipment has not been thoroughly investigated for sand aspha mixes. In one case in which it was tried the feet would not work o due to high moisture content or lack of cohesion in the mix but in anoth mixture containing less bitumen it appeared to work satisfactorily within about 1″ of the surface.

The finished compacted depth used in various states varies from 4 to 6″, although at least one project is 8″ thick. No experiments or labor tory tests have been made to show the theoretical depth which shoul be used. The depth is dependent on the subgrade, traffic, weather co ditions and the stability of the mix, and when all of these factors a considered, it would be very difficult to test for depth experimentall However, the depths in various states are proving satisfactory and i South Carolina 4″ compacted is used on light travelled roads and 5″ fo heavier traffic. It is entirely possible that greater economy can be a fected by varying depths even more than is now done.

"Sand Asphalt" is widely used not only as a base for new co struction but for patching, particularly repairing old surface treatme roads in which the soil base material is unsatisfactory. A covering

proximately 3″ of sand asphalt over old surface treatment has given cellent results in many installations and it appears to be the easiest and far the most economical method of repairing this type of road where ndy soil is available. One section of road through a river swamp was paired four years ago in this manner and since then repairs have en negligible. This particular road carries heavy truck traffic (about 0 per day) and the water table is only about two feet below the road rface, the subgrade is very plastic "gumbo" clay, and the road is bject to overflow during flood periods.

Sand asphalt mixtures are also used for shoulder treatment or widening of regular pavements and the small stationary mixing plant is particularly adaptable for this type of construction.

Due to the effort to keep this road as low as possible in initial cost, is usually necessary to use the soil most economically available and lapt the remainder of the process to the soil, but wherever possible e soil should be selected so as to secure the best graded material available with plenty of fine particles and one which will give the densest ssible mixture. There is wide latitude in grading of the soil, bitumen ntents, and methods of construction for this type of base, but there e certain fundamentals which must be observed if a sitisfactory job obtained. These are proper proportioning of the bituminous material, w moisture content for initial mixing, low moisture and naphtha content t time of compaction and thorough compaction. If these are controlled od results can always be expected. Laboratory tests are not as useful designing and controlling this type of construction as they should e, but this is due to the wide latitude which is possible in many of the equirements and to the effort to avoid cost refinements in the work.

GRADING OF TYPICAL COARSE AND FINE SAND USED IN CONSTRUCTION OF SAND ASPHALT BASES

Sand Clay Grading of Sand					
	Pass No. 20	Pass No. 60	Pass No. 100		Surface Area Sq. Ft. Per
et. No. 20	Ret. No. 60	Ret. No. 100	Ret. No. 200	Pass No. 200	100 lbs.
6.7	79.3	10.9	2.3	0.8	36.3
0.3	2.4	17.4	62.8	17.1	154.0

PULVERIZING AHEAD OF MIXING

MIXING MACHINE
(Front View)

MIXING MACHINE
(Rear View)

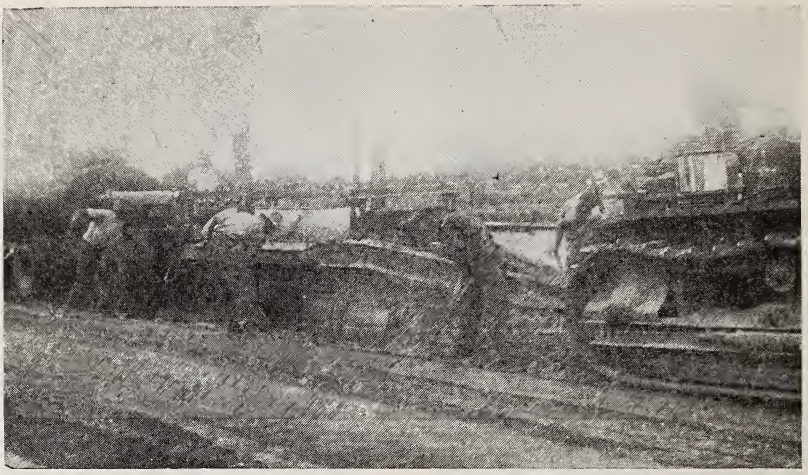
MIXED WINDROW

SPREADING AND AIRING

COMPACTING EDGES

COMPACTING

FINISHING



MIXING MACHINE

MIXING MACHINE

FINISHING

Role of the Swell Test in the Selection of Materials and Design of Dense Graded Bituminous Mixtures

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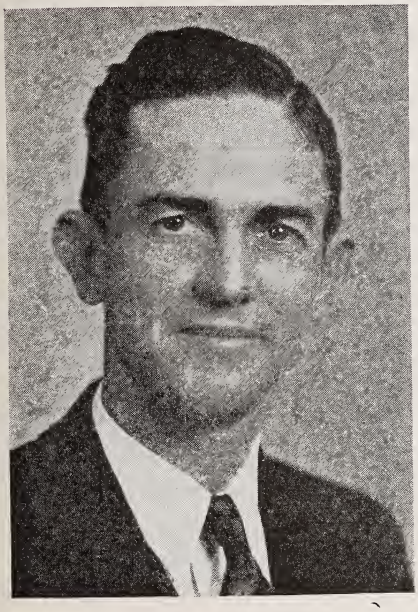

The swell test is a rapid and relatively simple test to determine the stability of a bituminous treated material subjected to the action of water. This study in fundamentals is based on comments made from time to time that oils from different sources and different methods of manufacture did not give the same results with the same mineral aggregate. It was also thought that there were other factors which might possibly effect the results of the test.

In the light of present day technic, it seemed desirable to follow up the expressed thought that results were not reproductible. E. F. Kelley, Chief of Division of Tests, Bureau of Public Roads, in a paper appearing in the Proceedings of the National Research Council* on stripping tests, makes the statement that "The test brought out a substantial difference in the behavior of the cracked and uncracked asphaltic materials. The cracked materials possessed much greater resistance to stripping than did the uncracked. The difference in this respect was very marked for

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the slow curing liquid asphaltic materials and to a lesser degree wi the cutbacks, emulsions and the penetration asphalts." The results this paper show the same trend.

The factors studied were:

1. Effect of length of time of immersion of the test specimen in water.
2. Effect of maximum particle size of the mineral aggregate upon the test results obtained.
3. Effect of oils produced from different types of crudes and subjected to different kinds of processing.

In order to get a cross-section of SC-2* oils produced and use throughout the United States and Canada, we requested oils from com panies in different areas. This request procured for us twenty-eigh samples of oil from the United States and one from Canada. The analyse of these oil samples are reported in Table 1.

Mineral aggregates used with these oils were from five sources whic actually have been used in constructing oil roads, and two from on source which had been turned down for use because of excessive swell Physical test results in these mineral aggregate are reported in Table 2

Sample 51603 represents mineral aggregate actually used in construc tion in 1928 on the Phoenix-Yuma highway between Hassayampa an Gillespie Dam (approximately 11 miles). The oil used was 60+ road oil the resulting mix emulsified badly as shown by Figure 1.

Sample 51604 represents mineral aggregate actually used in con struction in 1929 on the Globe-Safford highway extending from Globe t approximately six miles east. The oil used was 60+ road oil and th resulting mix emulsified badly giving the roadway almost identically th same appearance as shown in Figure 1.

Sample 51605 represents mineral aggregate actually used in con struction in 1930 on the Mesa-Superior highway beginning at Apache Junc tion and extending approximately 7 miles southeast. The oil used wa 60+ road oil. This material rapidly lost its life and, though it did not show signs of emulsifying, raveled badly.

Samples 51612 and 51613 represent a prospect of the Yuma-Gila Bend highway approximately five miles west of Gila Bend. It was not used on account of excessive swell of samples from prospect pits.

Sample 51614 represents mineral aggregate used in 1932 on the Florence-Tucson highway, beginning approximately ten miles southeast of Picacho and extending ten miles to the Pinal-Pima County line. The resulting mix emulsified slowly, even though cutback asphalt was used as a binder with the hope that it would prevent such action because original determinations showed excessive swell with 60+ road oil. The cutback used was specified as 100-200 asphalt cut with 18% to 21% kerosene.

Sample 51825 represents material first mixed in about 1930 and subsequently used with all type oils on many hundreds of miles with satisfactory results.

*See specification T101-38, Standard Method of Determining Swell of Bituminous Mixtures, adopted by A. A. S. H. O. which uses only SC-2 oils in this test.

PROCEDURE

Preparation of Sample—All samples were first screened over 1″ square, ¾″ square, ½″ square and No. 3 sieve. Then material passing No. 3 sieve was split to approximate quantity for each mix by means of a mechanical sampler and recombined to the grading shown in Table 2 for the 1 inch swell determination.

In preparing the samples of passing No. 10 material for test, the passing No. 3 material was split down to the approximate quantity required and then screened through a No. 10 sieve before mixing with oil. The required amount was weighed from this material. 1,500 gram samples of passing 1 inch material and 500 gram samples of passing No. 10 material were used for the two methods of determination of swell.

The entire series of samples of mineral aggregate were oven-heated to 105° C. and hand-mixed with each oil. The quantity of each oil added to the mineral aggregate was the same for a series of tests (see Table 2). After complete mixing, the sample was hand-tamped into the test ring and compressed against a steel plate by means of a plunger under a load of 2,000 pounds per square inch applied at the rate of 0.1 inch per minute. Three inch diameter rings, two inches high were used for the passing No. 10 material, and four inch diameter rings, four inches high were used for the passing 1 inch material. After compression, the plunger end (or bottom) of the test specimen was sealed with paraffine. The elevation of the top was then measured by means of an Ames dial supported on a tripod which rested upon the edge of the steel ring and readings were to the nearest .001 inch. After compression, sealing and measuring, the oil mix filled rings were immersed in tap water over night (16 hours). Each was then removed and the Ames dial reading again taken. Samples were then replaced in the water for 24 additional hours. This procedure was repeated until the completion of the test.

Sample 51604 was tested for swell on the passing No. 10 material only, as there was insufficient material for further work.

Sample 51614 was tested for sixteen hours only on the passing No. 10 material because at that time we had not realized the necessity of longer time immersion.

ANALYSIS OF RESULTS OBTAINED

1. **Effect of length of time of immersion of the test specimen in water.** Materials with very little swell show only slight increases in amount of swell for time periods greater than 16 hours as shown by aggregates 51605 and 51825—Table 3; and aggregates 51605, 51614 and 51825—Table 4.

Samples showing appreciable amounts of swell at 16 hours will show appreciable gain in swell at 40 hours, and somewhat less increase of swell for time periods of 64 and 88 hours. This effect is more noticeable in the passing No. 10 material than it is with passing 1-inch material. See samples 51603, 51604, 51612, 51613, Table 3; and samples 51603, 51612, 51613, Table 4.

2. **Effect of maximum particle size of the mineral aggregate upon the tes results obtained.**

Passing #10 sieve material produces higher swell than does passin 1″ material. However, either size material can be used in making swe tests by providing a proper specification limit to correspond to maximur size used.

3. **Effect of oils produced from different kinds of crudes and subjected t different kinds of processing.**

The source (geographically) and processing (by producer) of oil used in tests caused the most effect upon the swell. It may have bee fortunate in the original work on swells that oils from only one sourc (California) were available and as a consequence results were reproducible However, the composite picture as outlined here shows that results on th same mineral aggregate vary unless oils have certain characteristics.

It will be noted from aggregates 51605 and 51825, Fig. 4, and 51605 Fig. 6 and 51825, Fig. 7, that good aggregates show practically constan and very low swells with all the oils used. It will be noted however, tha extremely poor materials have the swell definitely diminished by som oils; see 51603, 51604, Fig. 2, 51612 and 51613, Fig. 3, and 51612, Fig and 51613, Fig. 6. An intermediate grade of aggregate, see 51614, Figs 4 and 7 shows some effect of swell diminution with certain oils.

It is also noted that an oil which diminishes the swell of one minera aggregate will have a similar effect upon other mineral aggregates how ever not always to the same degree. Oil samples 11, 7, 5, 20, 8, 2, and 1 show this diminution to a marked degree; samples 29, 21, 19, 6, 17, and 13 show consistently high swells.

The oil samples have been arranged in the tables and figures accord ing to decreasing Oliensis Xylene equivalent. It is a very evident fac that oils with an Oliensis Xylene equivalent equal to or greater tha 69% definitely diminish the swell. The reverse of this statement doe not hold true; however, it is true in most cases, the two exception being oil 1 and 12. These oils are both Oliensis negative and yet show diminution of the swell.

The general statement might be made at this point that an oil o less than 30 Xylene equivalent can be used in most cases in making th swell test to show unsatisfactory mineral aggregate.

Extractions were made to determine the type of oil that had bee used in the original construction of the projects on which these minera aggregates were used. Old samples of the completed mix stored in fric tion lid buckets were extracted with benzol and Oliensis Xylene equivalen and modified Oliensis were run on the residue from distillation to 680° F Swells were also run on these samples. The results are shown in Table 5

It will be noted that in all these samples the Xylene equivalent i less than 19%, showing there is little probability of swell diminution fro the oil. However, there is a possibility of some selective adsorption o those constituents of the oil which cause a high Xylene equivalent, thu not showing them in their true light. It is also noted that the swell

on the old retained samples are much less than on freshly mixed samples. These are phenomena of ageing or oxidation of some nature and an explanation will not be attempted here.

In concluding this paper, it seems advisable to make the following statements:

1. If the swell test is to be used as a basis for acceptance of mineral aggregate, the characteristics of the oil used in such testing will have to be specified.

2. Based on this data, the oil (SC-2) used in testing should have an Xylene equivalent of 30% or less.

3. The results of these laboratory tests indicate that the service behavior of questionable mineral aggregates may be modified by certain oils.

FIG. 1.

SHOWS EXTENT TO WHICH EMULSIFICATION TOOK PLACE ON MATERIAL REPRESENTED BY SAMPLE 51603. BACKGROUND SHOWS EFFECTIVENESS OF SEAL COAT IN PREVENTING INTRUSION OF WATER, THUS AIDING IN PRESERVING ROAD.

Fig. 2

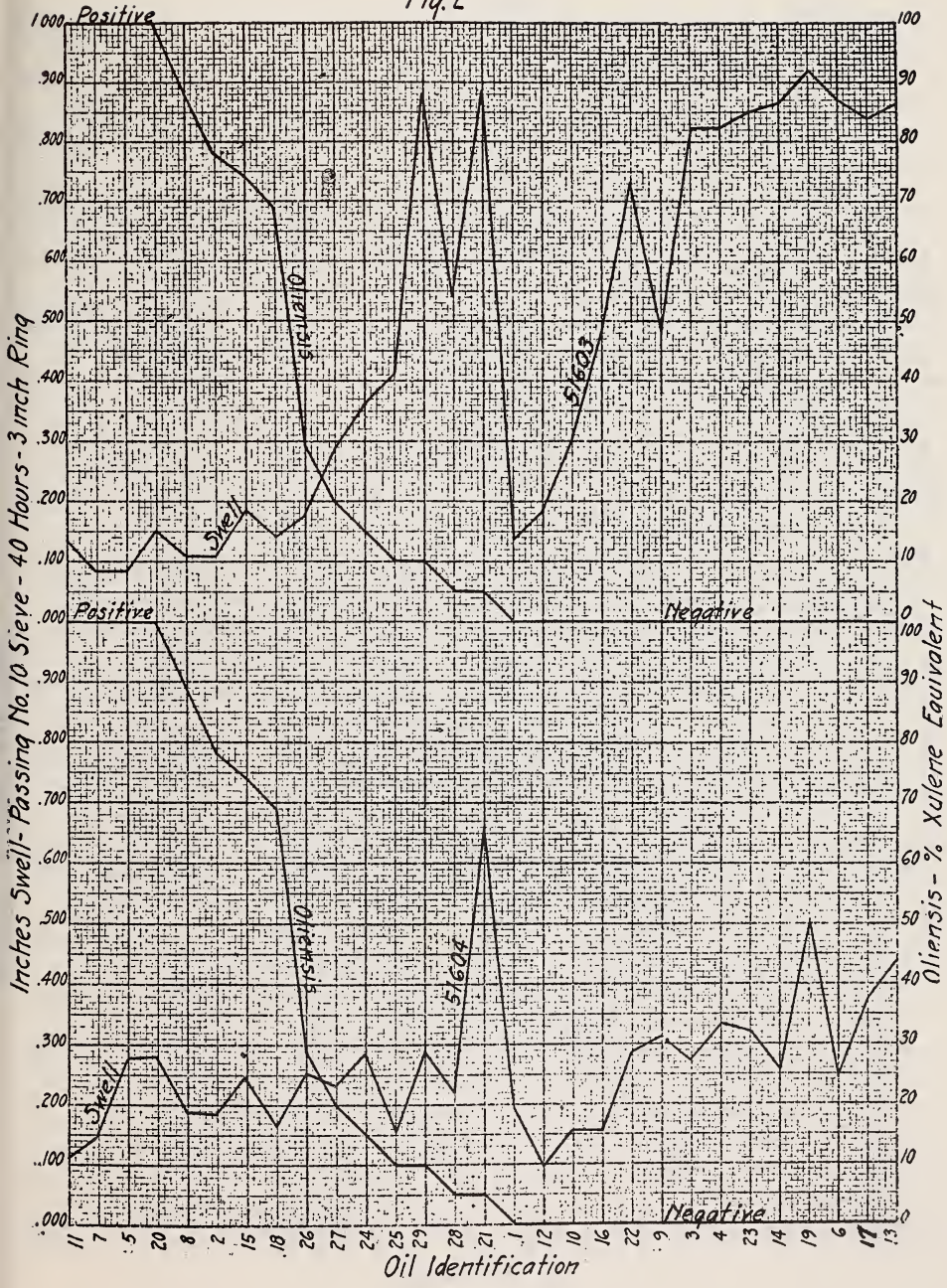
Positive
Negative
Positive
Negative
Swell
Oliensis
Swell
Oliensis
S1603
S1604
Inches Swell - Passing No. 10 Sieve - 40 Hours - 3 inch Ring
Oliensis - % Xylene Equivalent
Oil Identification
1.000
.900
.800
.700
.600
.500
.400
.300
.200
.100
.000
100
90
80
70
60
50
40
30
20
10
0
11 7 5 20 8 2 15 18 26 27 24 25 29 28 21 1 12 10 16 22 9 3 4 23 14 19 6 17 13

Fig. 3

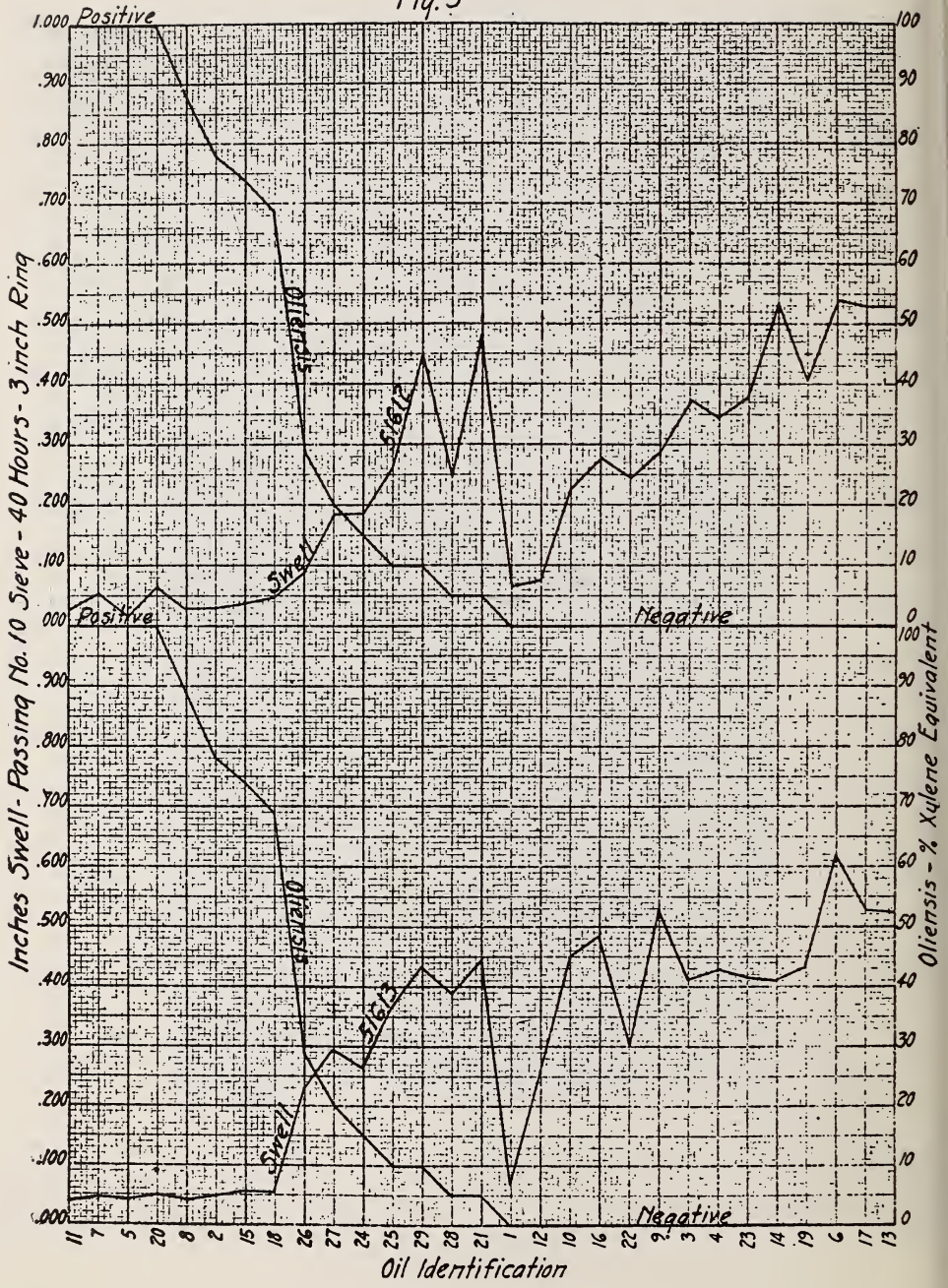
Positive
Negative
Swell
Sigma 10
Sigma 13
Inches Swell - Passing No. 10 Sieve - 40 Hours - 3 inch Ring
Oliensis - % Xylene Equivalent
Oil Identification
1.000
.900
.800
.700
.600
.500
.400
.300
.200
.100
.000
100
90
80
70
60
50
40
30
20
10
0
11 7 5 20 8 2 15 18 26 27 24 25 29 28 21 1 12 10 16 22 9 3 4 23 14 19 6 17 13

Fig. 4

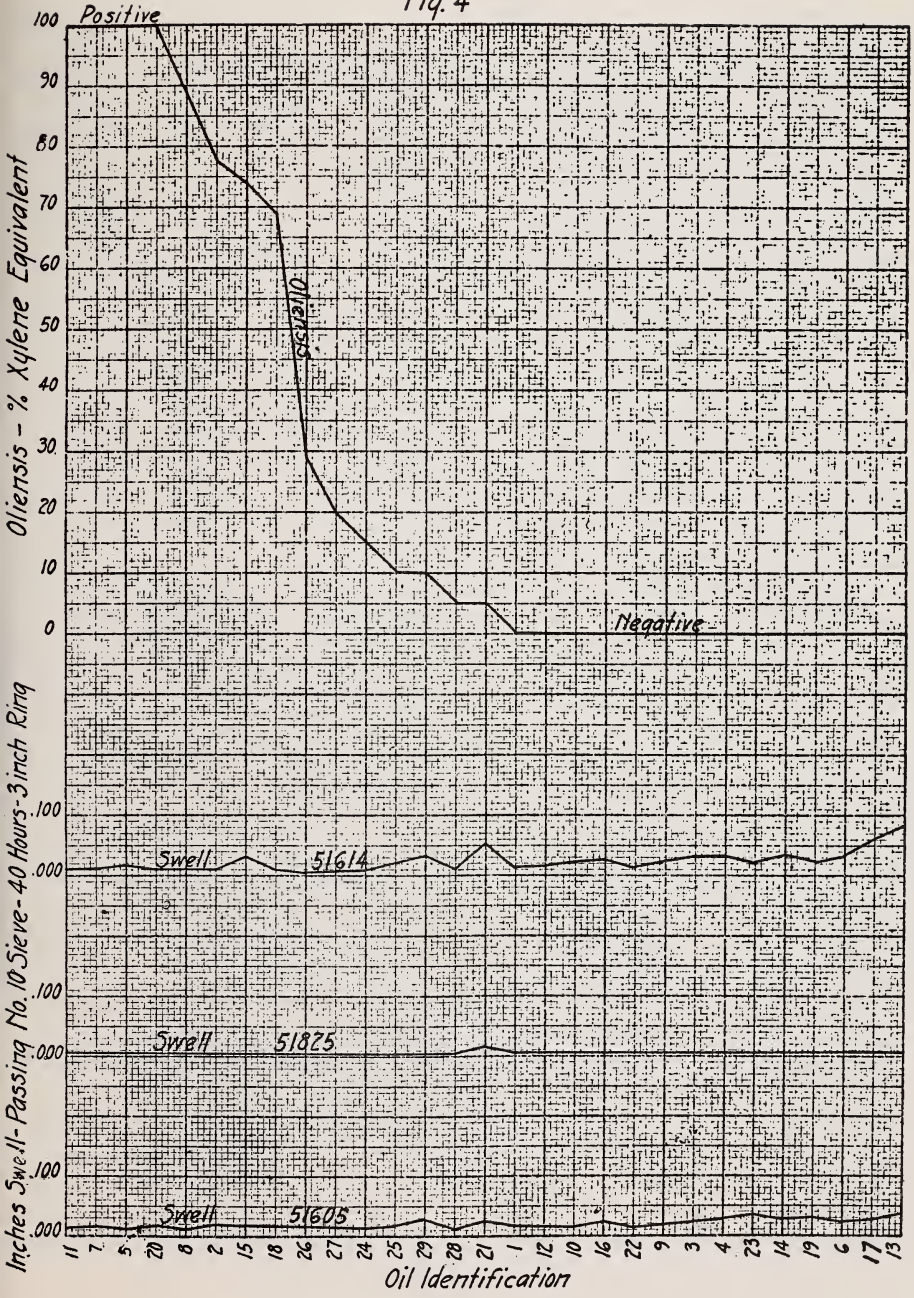
Positive
Negative
Oiliensis - % Xylene Equivalent
100
90
80
70
60
50
40
30
20
10
0
Inches Swell - Passing No. 10 Sieve - 40 Hours - 3 inch Ring
.100
.000
Swell
51614
.100
.000
Swell
51825
.100
.000
Swell
51605
11 7 5 20 8 2 15 18 26 27 24 25 29 28 21 1 12 10 16 22 9 3 4 23 14 19 6 17 13
Oil Identification

Fig. 5

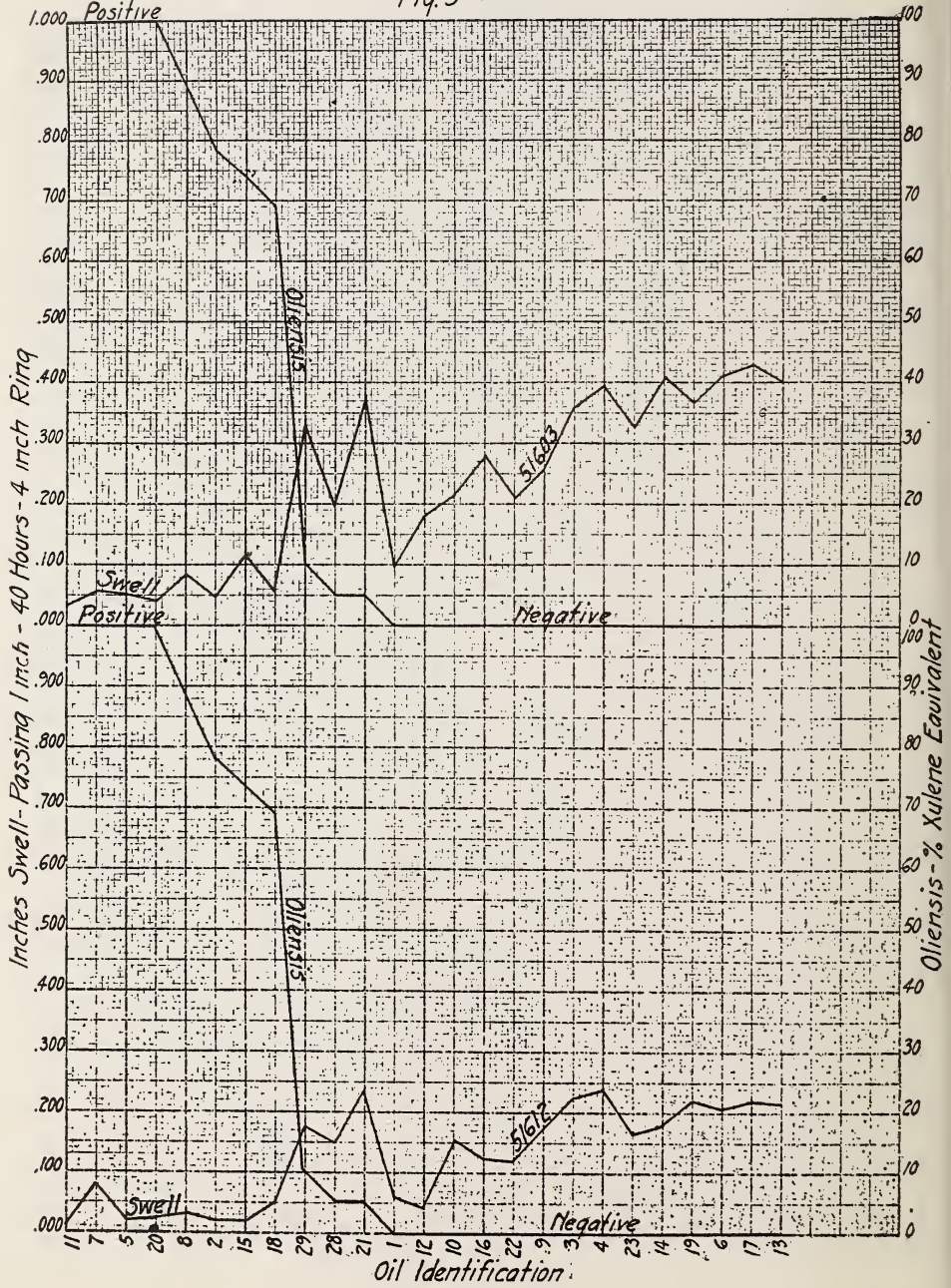
Positive
Oliensis
51643
Swell
Positive
Negative
Oliensis
51612
Swell
Negative
Inches Swell-Passing 1 inch - 40 Hours - 4 inch Ring
Oliensis-% Xulene Equivalent
Oil Identification
1.000
.900
.800
.700
.600
.500
.400
.300
.200
.100
.000
100
90
80
70
60
50
40
30
20
10
0
11 7 5 20 8 2 15 18 29 28 21 1 12 10 16 22 9 3 4 23 14 19 6 17 13

Fig. 6

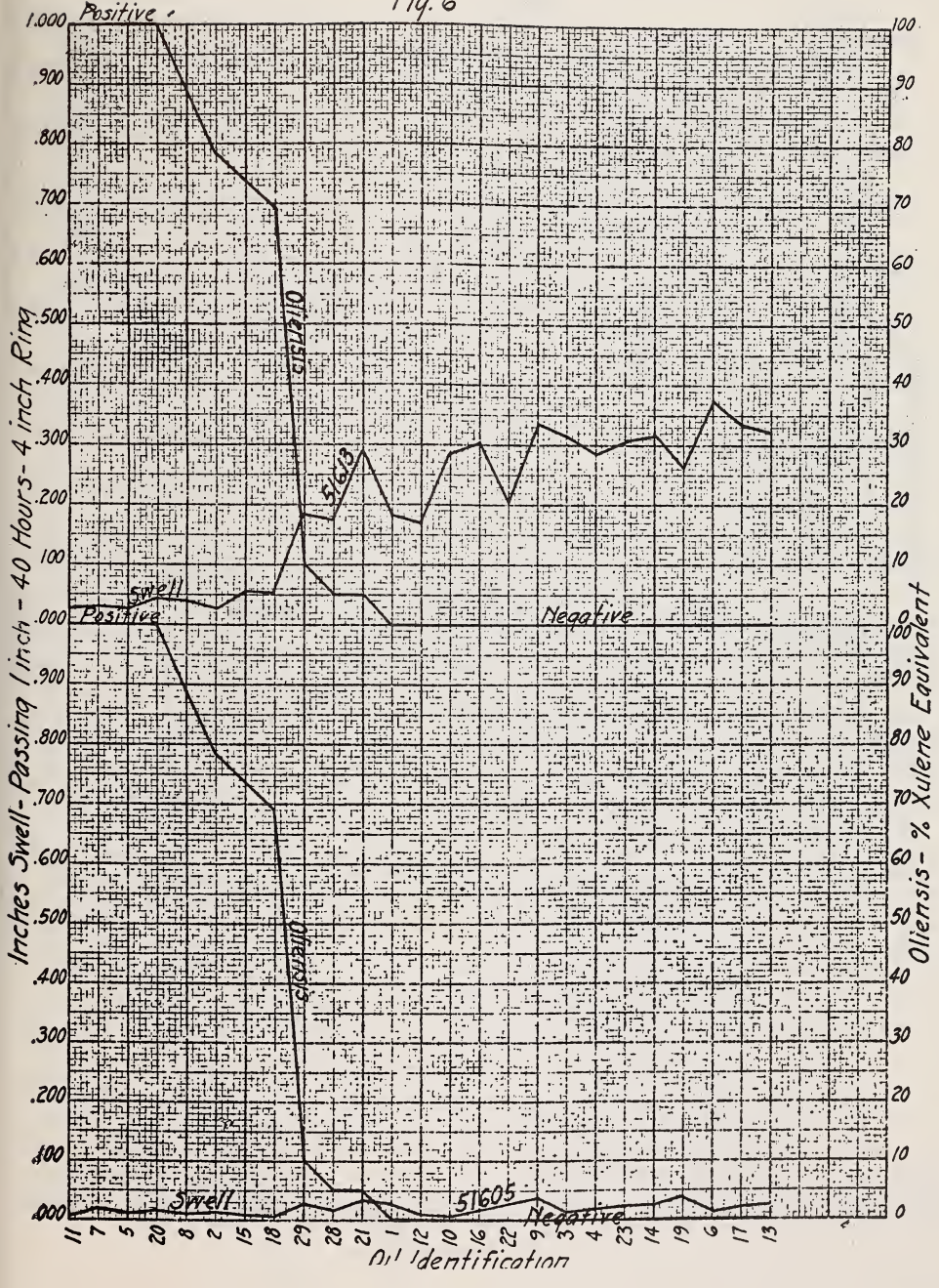
Positive
Swell
Positive
Negative
Swell
51605
Negative
Oil Identification
Inches Swell - Passing 1 inch - 40 Hours - 4 inch Ring
Oliensis - % Xulene Equivalent

Fig. 7

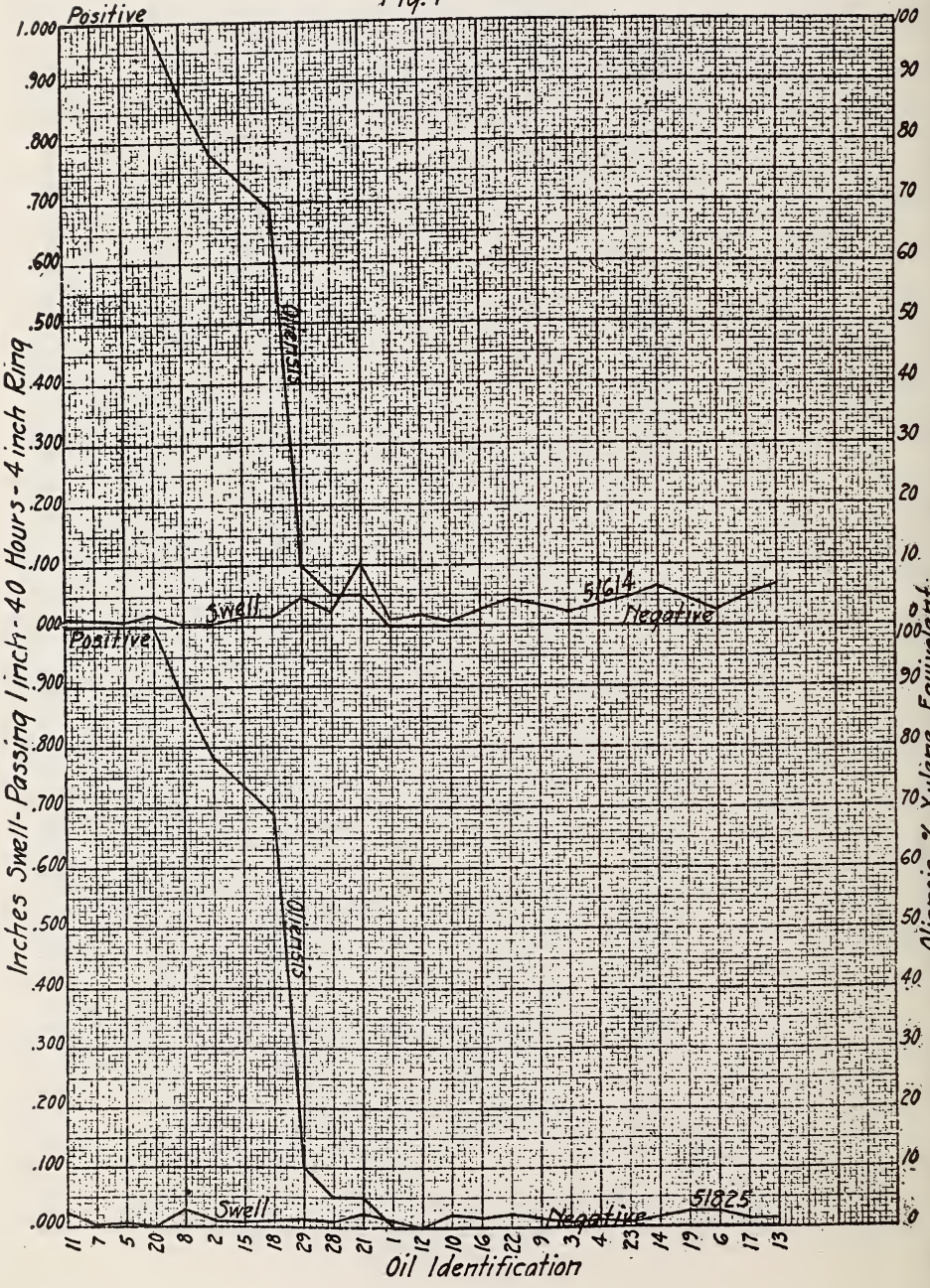
Positive
Swell
Negative
5164
51825
Oliensis
Inches Swell - Passing 1 inch - 40 Hours - 4 inch Ring
Oliensis - % Xulene Equivalent
Oil Identification
1.000
.900
.800
.700
.600
.500
.400
.300
.200
.100
.000
100
90
80
70
60
50
40
30
20
10
0
11 7 5 20 8 2 15 18 29 28 21 1 12 10 16 22 9 3 4 23 14 19 6 17 13

TABLE 1

	Source	Distillate to 600° F.	Distillate to 680° F.	Float Test of Residue at 122° F.	Solubility of Residue in CS_2	Asphalt of 80° Penetration	A. P. I. Gravity	Viscosity Saybolt-Furol at 122° F.	Float Test at 77° F.	Flash Point C. O. C.	Wax	Oliensis Spot Test	Xylene Equivalent	Modified Oliensis 35% Xylene	Average Swell 40 Hr. 3" Ring
Test Method		D402-36		D139-27	D4-27	D243-36	D287-37	D88-36	D139-27	D92-33	IPT 314	ASTM Prt. 2	Vol 36 Pg. 494	*	
SC-2	Spec. Max	15%	25%					200	200						
	Min.			25 Sec.	99.0	65%		320		200					
Oil No.															
11	West Kansas	0	2.00	35	99.93	68.42	4.7	184	42	340	0.89	+	100+	+	.047
7	Cracked Illinois	0	1.00	35	99.99	67.88	5.9	193	46	365	1.44	+	100+	+	.050
5	Cracked Mid. Cont.	2.50	11.25	86	99.89	73.41	4.7	181	48	265	1.53	+	100+	+	.064
20	Cracked	Tr.	6.50	66	99.93	74.38	3.5	317	65	290	0.28	+	100+	+	.083
8	Cracked W. Tex. & Mid. Cont.	1.00	9.25	59	99.92	69.87	6.0	163	46	275	1.00	+	83-88	+	.055
2	Cracked E. Kentucky	0	5.75	40	99.96	71.84	8.3	281	45	310	0.97	+	74-78	+	.058
15	Wyoming	3.50	19.50	112	99.98	71.32	9.3	239	37	275	1.08	+	69.74	+	.082
18	Cracked Oklahoma	0.50	7.75	52	99.83	72.52	8.7	245	51	275	1.04	+	64.69	+	.061
26	Del Rey Calif.	0.50	7.50	62	99.93	68.33	12.9	312	70	275	1.40	+	26-29	+	.109
27	Wilmington & Del. Rey. Calif.	0.75	9.50	56	99.92	66.98	13.3	290	55	260	1.20	+	16-20	+	.146
24	Calif	1.00	9.50	60	99.90	67.00	13.3	310	53	255	1.13	+	11-15	—	.161
25	Wilmington Calif.	Tr.	9.50	45	99.91	61.71	13.6	327	34	280	0.81	+	6-10	—	.177
29	Calif.	10.25	19.25	134	99.97	72.40	14.2	305	55	185	0.66	+	6-10	—	.302
28	Mixed Calif.	1.00	12.00	61	99.87	67.54	13.8	310	41	265	0.99	+	1-5	—	.203

TABLE 1—(Continued)

Oil No.	Source	Distillate to 600° F.	Distillate to 680° F.	Float Test of Residue at 122° F.	Solubility of Residue in CS2	Asphalt of 80° Penetration	A. P. I. Gravity	Viscosity Saybolt-Furol at 122° F.	Float Test at 77° F.	Flash Point C. O. C.	Wax	Oliensis Spot Test	Xslene Equivalent	Modified Oliensis 35% Xylene	Average Swell 40 Hr. 3" Ring
Test	Method	D402-36		D139-27	D4-27	D243-36*	D287-37	D88-36	D139-27	D92-33	IPT 314	ASTM Prt. 2	Vol 36 Pg. 494	*	
SC-2	Spec. Mar	15%	25%					200	200						
	Min.			25 Sec.	99.0	65%		320		200					
21	Uncracked	7.50	20.00	83	99.95	68.54	15.4	313	31	240	0.81	+	1-5	—	.367
1	Unknown	0	11.00	77	99.90	71.59	17.5	298	61	300	1.20	—	0	—	.070
12	West Kansas	0	2.00	42	99.88	67.60	9.0	277	56	335	1.03	—	0	—	.091
10	Kansas	0	14.50	49	99.88	66.94	15.6	186	21	315	1.70	—	0	—	.168
16	So. Calif.	2.00	15.50	70	100.00	69.72	14.6	269	34	265	0.50	—	0	—	.207
22	Mixed Calif.	6.50	16.50	81	99.90	68.06	13.9	304	46	200	1.35	—	0	—	.288
9	Ventura & Coalinga, Calif.	Tr.	9.50	49	100.00	67.57	14.5	273	42	270	1.07	—	0	—	.235
3	Venezuela	9.75	17.50	90	99.98	70.48	14.6	233	78	230	1.01	—	0	—	.277
4	Los Angeles	0.50	12.75	50	99.99	64.61	13.8	251	35	270	0.60	—	0	—	.285
23	San Joaquin Calif.	3.00	13.75	78	99.94	68.11	13.8	285	31	245	0.23	—	0	—	.289
14	Wyoming	1.25	15.50	65	99.95	67.29	12.6	297	35	295	0.93	—	0	—	.302
19	Unknown	0.50	8.00	35	99.94	64.55	12.3	268	25	275	0.50	—	0	—	.332
6	Smackover Arkansas	6.00	15.00	68	99.99	72.02	13.9	276	32	235	1.09	—	0	—	.334
17	Venezuela	0.75	10.50	41	99.96	67.05	14.0	303	25	280	0.35	—	0	—	.340
13	Unknown	16.50	26.00	172	98.97	68.99	13.2	245	25	195	0.22	—	0	—	.355

*Modified Oliensis=Same as oliensis except 10.0 ml. solvent and 2 gr. oil ± .02 gr., used.
Solvent consists of 35% Xylene and 65% n-Heptane of knock rating grade.

TABLE 2

Sample No.—	51603	51604	51605	51612	51613	51614	51825
Project No. —	64A	87C	93B	56	56	94D	Salt River
SIEVE ANALYSIS AS USED							
Retained on Sieve No.							
1″	0.00	0.00	0.00	0.00	0.00	0.00	0.00
¾″	6.67	6.67	6.67	6.67	6.67	6.67	6.67
½″	13.33	13.33	13.33	13.33	13.33	13.33	13.33
3	20.00	20.00	20.00	20.00	20.00	20.00	20.00
4	6.12	7.44	3.72	4.96	2.28	6.72	3.72
8	12.64	14.17	8.72	8.82	5.88	13.18	5.92
10	3.42	2.76	2.70	1.76	1.76	4.26	1.56
16	7.82	6.72	8.78	4.82	5.84	9.74	4.54
30	8.60	6.70	16.14	11.64	15.12	9.72	10.12
50	6.38	6.16	12.54	15.82	17.83	7.98	14.64
100	2.78	4.65	4.78	4.92	5.70	2.98	11.40
200	1.56	2.00	1.14	1.92	1.34	1.26	5.46
Pass 200	10.68	9.40	1.48	5.34	4.25	4.16	2.64
ANALYSIS MATERIAL PASS NO. 40							
Liquid Limit	36	43	29	25	31	28	21
Plastic Limit	23	23	0	0	27	19	0
Plastic Index	13	20	0	0	4	7	0
Field Moisture Equivalent	26	35	24	24	30	23	19
Centrifuge Moisture Equivalent	19	27	7	7	8	13	4
Shrinkage Limit	24	24	22	21	27	19	19
Shrinkage Ratio	1.6	1.6	1.5	1.6	1.5	1.7	1.6
Field Volumetric Change	4	18	4	4	4	7	1
Lineal Shrinkage	1.3	5.3	1.3	1.4	1.3	2.2	0.3
ANALYSIS MATERIAL PASSING NO. 10							
Moisture Equivalent (Rose)	21.0	25.4	21.2	19.0	21.2	14.4	15.8
Lineal Shrinkage (Rose)	1.56	4.17	0.52	0.52	0.52	0.52	0.00
PERCENT OF OIL ADDED TO MAKE SWELL							
Material Passing No. 10 Sieve	13.0	14.00	7.00	8.00	10.00	8.00	8.00
Material Passing 1″	5.33		4.00	4.33	5.33	4.67	

TABLE 3

SWELL TEST — MATERIAL PASS NO. 10 SIEVE — 3″ RING

Oil No.	Aggregate No. 51603 16 Hr.	40 Hr.	64 Hr.	88 Hr.	Aggregate No. 51604 16 Hr.	40 Hr.	Aggregate No. 51605 16 Hr.	40 Hr.	64 Hr.	Aggregate No. 51612 16 Hr.	40 Hr.	Aggregate No. 51613 16 Hr.	40 Hr.	Aggregate No. 51614 16 Hr.	Aggregate No. 51825 16 Hr.	40 Hr.
11	.072	.131	.192	.218	.069	.109	.007	.011	.016	.010	.025	.035	.038	.011	.001	.003
7	.083	.083	.133	.151	.106	.144	.008	.013	.016	.030	.053	.034	.042	.012	.001	.005
5	.048	.088	.152	.186	.116	.272	.007	.010	.015	.007	.014	.033	.039	.020	.002	.004
20	.093	.149	.156	.168	.190	.277	.014	.016	.017	.033	.067	.048	.051	.013	.002	.007
8	.062	.110	.198	.238	.117	.184	.007	.010	.014	.022	.028	.035	.038	.012	.003	.004
2	.071	.108	.178	.232	.132	.184	.018	.018	.021	.025	.030	.039	.047	.012	.004	.008
15	.096	.184	.209	.243	.149	.244	.014	.016	.019	.022	.035	.043	.058	.034	.002	.005
18	.120	.140	.158	.176	.068	.165	.013	.015	.018	.035	.038	.052	.052	.011	.001	.005
26	.152	.177	.210	.232	.173	.257	.008	.013	.017	.076	.086	.207	.222	.006	.001	.004
27	.243	.288	.342	.372	.115	.231	.009	.013	.017	.135	.182	.292	.294	.009	.000	.005
24	.290	.363	.381	.399	.155	.289	.011	.012	.020	.122	.184	.260	.264	.011	.000	.005
25	.412				.055	.153	.015	.015	.020	.248	.261	.369		.022	.000	.005
29	.882				.162	.289	.021	.024	.033	.450		.431		.033	.002	.004
28	.542				.065	.219	.011	.011	.018	.238	.248	.386		.013	.001	.004
21	.881				.665		.026	.026	.030	.482		.444		.051	.006	.013
1	.099	.133	.223	.256	.127	.191	.018	.018	.018	.041	.063	.059	.066	.015	.003	.006
12	.139	.181	.225	.253	.053	.092	.012	.015	.018	.051	.073	.244	.251	.017	.004	.006
10	.258	.305	.360	.376	.084	.155	.015	.015	.021	.222	.222	.450		.021	.002	.005
16	.480				.093	.153	.020	.021	.026	.236	.278	.486		.027	.002	.005
22	.735				.174	.288	.010	.013	.017	.186	.239	.300		.018	.003	.005
9	.483				.179	.310	.015	.019	.022	.282	.287	.523		.021	.003	.004
3	.822				.168	.271	.020	.021	.022	.373		.410		.032	.004	.007
4	.823				.202	.335	.026	.026	.027	.346		.428		.033	.002	.007
23	.848				.173	.322	.029	.032	.042	.380		.413		.022	.007	.009
14	.862				.141	.250	.019	.024	.031	.532		.406		.035	.001	.005
19	.923				.332	.508	.032	.028	.036	.401		.429		.027	.004	.010
6	.872				.114	.248	.019	.022	.027	.539		.615		.037	.005	.007
17	.839				.376		.035	.029	.033	.531		.529		.068	.003	.007
13	.861				.440		.029	.033	.033	.532		.532		.082	.004	.006

TABLE 4

SWELL TEST — MATERIAL PASS 1" - 4" RING

Oil No.	Aggregate No. 51603 16 Hr.	40 Hr.	88 Hr.	Aggregate No. 51605 16 Hr.	40 Hr.	Aggregate No. 51612 16 Hr.	40 Hr.	64 Hr.	Aggregate No. 51613 16 Hr.	40 Hr.	Aggregate No. 51614 16 Hr.	40 Hr.	88 Hr.	Aggregate No. 51825 16 Hr.	40 Hr.
11	.019	.033	.056	.002	.007	.009	.016	.019	.017	.025	.006	.012	.019	.007	.024
7	.042	.057	.080	.014	.020	.078	.083	.108	.022	.030	.009	.011	.013	.004	.004
5	.031	.050	.077	.007	.009	.012	.017	.023	.020	.026	.008	.005	.022	.007	.007
20	.026	.039	.077	.036	.015	.010	.025	.022	.024	.042	.012	.018	.024	.000	.000
8	.055	.079	.089	.010	.009	.020	.032	.032	.030	.039	.005	.007	.018	.020	.026
2	.032	.047	.085	.005	.012	.008	.020	.027	.028	.028	.000	.003	.010	.010	.011
15	.178	.113	.131	.009	.007	.036	.018	.026	.042	.057	.013	.016	.022	.007	.008
18	.134	.053	.073	.004	.003	.038	.053	.068	.036	.057	.005	.016	.016	.010	.011
29	.334			.030	.027	.181	.176	.190	.184		.045	.048	.048	.012	.013
28	.191	.196	.276	.020	.015	.155	.150	.172	.173		.019	.024	.029	.008	.010
21	.361	.373	.391	.028	.032	.221	.237	.226	.290		.058	.102	.124	.016	.026
1	.066	.096	.131	.013	.024	.034	.057	.054	.181		.005	.008	.018	.013	.008
12	.168			.011	.010	.031	.044	.046	.170		.016	.020	.020	.000	.000
10	.211			.004	.003	.140	.160	.162	.287		.004	.006	.029	.006	.020
16	.283			.013	.014	.122	.127	.142	.301		.024	.024	.035	.014	.015
22	.205	.213	.243	.020	.027	.110	.120	.125	.204		.029	.042	.047	.008	.021
9	.256	.376		.007	.038	.158	.173	.161	.335		.028	.032	.048	.007	.017
3	.357	.371		.005	.012	.199	.224	.219	.314		.015	.026	.046	.009	.009
4	,389	.387	.382	.020	.018	.214	.238	.219	.287		.028	.031	.038	.011	.013
23	.323			.020	.023	.163	.168	.163	.304		.043	.046	.055	.008	.009
14	.410	.400	.409	.023	.025	.180	.180	.168	.316		.062	.063	.066	.014	.015
19	,356	.356	.386	.036	.038	.245	.220	.224	.261		.046	.049	.048	.017	.027
6	.390	.414	.423	.016	.015	.202	.203	.192	.376		.021	.026	.043	.023	.023
17	.434			.024	.022	.223	.218	.203	.337		.047	.050	.060	.013	.013
13	.400	.412	.402	.031	.025	.195	.215	.220	.321		.065	.065	.075	.017	.016

TABLE 5

OLIENSIS TESTS ON DISTILLATION RESIDUE OF OIL EXTRACTED FROM OLD SAMPLES

Material From Same Source as Sample No.	Oliensis	Xylene Equivalent	Modified Oliensis	Swell - Passing No. 10 Sieve
51603	+	10-15	—	0.088
51603	+	10-15	—	0.114
51604	+	15-20	—	0.026
51604	+	25-30	—	0.032
51605	+	35-40	+	0.011
51605	+	15-20	—	0.014
51614	—	0	—	0.009*
51614	—	0	—	0.007*

*Cutback asphalt in mix.

Dense Graded Bituminous Mixtures Factors Influencing the Design

By

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Having started the construction of bituminous surfaces in 1927, we have now completed eleven years of experience. During that time we have constructed many different types, ranging from thin mats of SC oils with little or nor bases, through the heavier MC mats and up to two inch penetration macadam surfaces on heavy crushed rock bases using as a binder 95+ residual oil. We point with pride to the fact that, although, in some instances maintenance costs have been high, not one of these projects has ever reverted to a surface condition of a lower type.

I shall endeavor to trace briefly our experiences with the various types, give our reasons for the selection of the binder used, point out our failures encountered and the reasons and then lead up to our present design principles which have evolved from our past experiences. Funds for research have been practically nil and what progress we have made in the construction of bituminous surfaces has been largely the result of experience. This condition is gradually changing however, and much closer control is now being exercised by the laboratory.

The thin mats built in the early days of oil construction were, of course, necessary since many miles had to be built with our limited funds. In many instances only sufficient gravel was present on the roadway to build mats of one inch or less in thickness, leaving none for a base. The clamor for more miles of oiled surfaces, however, made it expedient on the part of our Highway Officials to oil what was left on the surface. Since even a fair oiled surface is far superior to a dusty gravel road the course taken was fully justified. Moreover, the failures experienced with these thin mats have been an excellent guide to later construction.

At that time the only type of binder available was a residual oil having approximately 60% of 100 penetration residue. Grades were narrow and the mats had to be extended practically to the slope line leaving little or no support for the mat edges. The most common failures were shoving and rutting at the edges causing depressions into which water settled and finally found its way through the mat to saturate the soil below and destroy, still more, its stability. Corrugating of the surfaces also occurred consistently, because of over rich mixtures and wet subgrades. Some trouble was experienced with surface ravelling but this was quickly checked by the use of plain seal coats.

Oil was applied to the gravel on the road surface directly by means of pressure distributors. It was practically impossible to maintain con-

stant speeds up and down long grades which resulted in lean and rich spots in the finished surfaces. Travelling plants have eliminated this condition under present practices and by keeping the percentage of oil relatively low, the tendency toward corrugating of the surfaces has been definitely corrected. Thorough seal coating of all projects has eliminated the infiltration of surface waters which had the effect of destroying, to some extent, the binding power of the oil as well as the weakening of the supporting power of the base and subgrade.

Soon after the advent of the medium curing types of cutbacks, this department launched an extensive program of bituminous mat construction using MC-3 as the binding medium. We fully expected to produce a surface far better in all respects than we had been able to with the residual oils. Slow curing oils were definitely eliminated and only a few projects were built using RC-3 as a binder.

Three years experience with the MC-3 binder has fully demonstrated to us that the type of oil for a given project should not be arbitrarily set but rather that all conditions should be considered before making a definite decision. We have found that MC-3 when used as a binder develops a harder, tougher mat and one which is less susceptible to moisture. While not as critical as regards the correct proportioning of oil and aggregate, we have had a few failures develop because of using excessive amounts of MC-3. This type of binder lends itself ideally to the construction of crushed rock mats especially where relatively small percentages of fines are developed in the crushing process. Our practice, in the building of this type of mat, is to mix the crusher material—¾″ maximum size—without adding any filler. After the mat is laid and rolled, a sand seal is placed on the surface in such amount as to thoroughly and tightly seal against surface water. An excellent mat is developed in this manner and a considerable saving of oil is effected by eliminating the use of a filler.

Practically all of our aggregates are developed from roadside deposits which in a great many instances contain high percentages of fines These materials lend themselves readily to SC type of mat construction, the fines and the resultant dense grading developing a stability with a lighter binding medium fairly comparable to that obtained with the heavier cutbacks and crushed rock.

Much attention is given to the average annual precipitation in the area of a given project before a decision as to the type of binder is made. If the mat is to be subjected to heavy winter snows and high rainfall, the decision is usually in favor of MC-3, since our experience has shown that such mats are less subject to damage from this source. Swell tests and stability tests of the aggregate to be used are also made using various types of binder and the results of these tests are likewise considered in making the final decision. If the decision otherwise is in favor of MC-3, however, the latter tests usually confirm the decision since as a rule a given aggregate will show less tendency to swell and will develop more stability when mixed with MC-3 than when mixed with SC-3.

A few projects have been constructed in which we used MC-2 as

the binding medium but this type of cutback proved entirely unsatisfactory because of its lack of cementitiousness. The mats develop brittleness, broke up badly and the mixture appeared "lifeless."

RC-2 has also been used with a few crushed rock mats and has proven entirely satisfactory. Since, however, equally satisfactory mats can be developed with crushed rock and MC-3, the latter type of binder is now used exclusively for this type of construction, because it is much easier to handle from a construction standpoint.

Several years ago a number of projects were built by the plant mix method but this was abandoned in favor of the road and traveling plant mix since the quality of the finished job did not seem to warrant the additional cost. SC-4 was used exclusively as a binder in all of these projects.

For the first time, we are this season using SC-4 with gravel aggregate on one project to determine whether or not it can be successfully handled in the travelling plant and road mix construction. Results indicate that it can be mixed with very little more difficulty and we will very likely specify this grade when slow curing oils are to be used as the binder.

Summarizing, then, as regards the types and grades of liquid asphalts used and our reasons therefore, we find that experience has lead us to the following conclusions. MC-3 has proven more satisfactory for the more open type of crushed rock mat due to its better binding quality in the absence of finer sizes in the aggregate. For the same reason it has proven more effective with the fairly dense graded crushed gravel types which lack the higher percentages of minus 200 mesh. Due to the fact that it offers more resistance to the stripping action of water it has been our choice for projects in those areas of heavy precipitation which are for the most part in our mountainous regions. Border line cases, where either MC to SC might be used to equal advantage are decided on the basis of the swell and stability tests.

The SC-3 type of oil has proven its value with the dense graded type of construction under conditions far less favorable than those under which it is now being used and we therefore have no hesitancy in specifying its use when the aggregate grading is such as to justify this grade. Mixes using SC are kept definitely on the lean side since our experience has shown that greater stability is thus developed and when thoroughly sealed against surface water less shoving and rutting occurs. It is quite likely that SC-4 will be substituted for SC-3 in much of our work in the future. The SC oils have the advantage of being less costly also.

Our experience with other types and grades of oils and cutbacks has satisfied us that for our work the MC-3 and SC-3 and 4 are the only ones we need consider. All projects are given a final seal coat and for this we have found RC-2 and a special grade designated as RC-1A to be most satisfactory. The latter contains more volatile diluent than the regular RC-1 grade and has a lower penetration residue. It is used exclusively for sealing SC mats when no cover coat is applied.

The RC-2 is used in sealing all MC mats both with cover and without. Stone chip seals are being specified on many of our recent jobs. Past experience here again has demonstrated that the life of a seal of this type justified the added cost. The non-skid surface developed is furthermore an added safety feature.

Much comment has recently been made regarding the hardening of liquid asphalts in service. We have checked a few of our projects and find this to be true. Samples of the mixed material are now being forwarded from all projects for extraction and analysis of the oil. Additional samples will be taken from the same locations from time to time so that we may determine the extent of hardening of the oil. If we find that progressive hardening is taking place we expect to take steps to improve conditions, probably by requiring that all MC products used for mixing be made softer base asphalts.

As a result of past experiences many changes have been made in our present designs. Much attention is being given to the construction of grades. Soil samples are taken during preliminary survey of all projects and data is obtained in the laboratory as to classification, optimum moisture and density. Charts are furnished the engineers to guide them as to the correct amount of moisture and rolling necessary to obtain ideal compaction. Follow up tests are made during construction to assure densities which are at least equal to 90% of that obtained by standard laboratory methods. Sheeps foot rolling of the outer third on each side of all projects is required. Compaction of the center third of the roadway is accomplished by directing thereon as much hauling as possible with the heavy equipment operating on the project. When necessary some additional rolling with sheeps foot rollers is also used. Special attention is given to the shoulder rolling on all high fills. Slopes on fills are built as follows — up to 3′-5 to 1, 3′ to 6′-4 to 1, 6′ to 10′-2 to 1 and above 10′-1½ to 1.

Road beds on our class 1 roads are 36′, on class 2 roads they are 30′ and on secondary roads 26′. Base course of both pit run and crushed gravel are constructed in courses varying in thickness from four inches on A1-2 and 3 subgrade soils up to 18 inches on the poorer types of soil. These bases extend from shoulder to shoulder, the trench type of base construction with dirt shoulders, having been entirely eliminated. All bases are soil bound, watered, processed, shaped and rolled.

Mats on our 36′ road sections are built to a width of 24′ leaving a 6′ gravel shoulder section on each side of the mat. The wide grade with flat slopes and the gravel shoulders afford ample side support and should definitely eliminate our mat edge failures. 22′ mats are placed on both the 30′ and 26′ road sections. Experience has demonstrated this design to be satisfactory provided the grade is constructed of good soils but more width of grade and at least four feet of gravel shoulder on each side of the mat should be provided to give the necessary support when subgrade soils classify below an A-3 type. One recent project has definitely proven this point. The grade, in which an A-6 soil predominated, was built to a 26′ width. The gravel base of 6″-4½″-6″ section was placed with the same slope as the grade which resulted in a width at top of gravel

section of approximately 22′. The mat was designed for 22′ thus leaving no shoulder to support the mat edge. Melting snows and spring rains seeped back under the mat for a distance of about two feet thus weakening both grade and base course. Severe cracking of the mat has occurred for this distance of two feet on each side. The balance of the surface is in excellent condition. Traffic on this road is extremely light and could have by no means, contributed to the failure. Thorough compaction of the grade and wider grades and shoulders are thus shown to be essential wherever the poorer types of soils must be used.

The new design, especially that used on our class 1 roads, with wide well compacted grades, heavy well bound bases and wide mats with adequate shoulders gives us a finished roadway that contrasts most favorably with anything we have heretofore built. We are confident that most of our past failures have definitely been eliminated and that we have added immeasurably to the safety of our highways.

"Open-Graded Type of Bituminous Construction" Factors Influencing the Selection of Aggregate and Bituminous Binder

By

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During the glacial era nearly all of what is now the State of Minnesota was covered with a great ice sheet that had moved down from the Canadian highlands carrying with it quantities of rock and soil. When the ice melted it segregated large quantities of sand and gravel material which were deposited to form eskers, kames and outwash plains. These deposits are numerous and are found in almost every portion of the state and consequently are a very valuable asset in supplying low cost aggregate for road building purposes. Such aggregate, in addition to being of relative low cost, consist almost entirely of sound durable particles of high quality.

In order that such readily available local aggregate might be used, the Minnesota Highway Department has employed a mixture design which would be favorable to its use. Practically all Minnesota gravel deposits contain little material finer than the No. 200 mesh sieve, the percentage usually being less than four per cent. In other words, our natural deposits produce relatively clean open-graded aggregates having a void content of probably from 20 to 30 per cent.

The following table (Table 1) illustrates the grading of a typical Minnesota gravel aggregate and gives also the specification limits for aggregate for both plant-mix and road-mix construction.

TABLE 1

Sieve Size (Square Opening)	Typical Minnesota Gravel* Per Cent Passing	Specification Requirements** Plant-Mix Gravel Aggregate Per Cent Passing	Road-Mix Gravel Aggregate Per Cent Passing
¾	100	100	100
⅝	97	95-100	95-100
⅜	88	65–95	65–95
No. 4	60	45–70	—
No. 10	40	35–45	25–60
No. 40	20	5–25	5–35
No. 200	3	0–7	0–7

*Crushed to ¾ inch maximum size.

**Our present specifications specify a maximum loss in the Los Angeles Rattler test of 35 per cent and with not more than 15 per cent of particles showing a loss in excess of 50 per cent. The percentage of shale and other unsatisfactory materials is limited to five per cent.

To utilize such aggregates in low cost road construction, two alternatives suggest themselves. The gravel aggregate could be supplemented with non-expansive soil fines so that the voids could be materially reduced with resultant increase in stability. With such a gradation it would be possible to use slow-curing oils as the bituminous materials. In this method it would be necessary to carefully control the mixture and proportion of the soil fines and gravel aggregate which although not a serious problem in plant-mix work does introduce some difficulties in the case of road-mix construction.

The other alternative is to use the gravel aggregate as found, crushing the same to a maximum size of ¾ inch. In such event it is necessary to use bituminous material which would lose a volatile fraction leaving a heavy viscous binder thus contributing to the stability of the mixture. This method would require less nicety of control of the quantity of bituminous material because of the mixture being less critical to bitumen content. In view of the above, it has been the policy of the Minnesota Highway Department to adopt the second procedure and it has worked out quite satisfactorily. To date there has been no displacement of mat which could be attributed to faulty grading or excess bituminous materials. Such displacement as has occurred could be attributed directly to instability of the base.

For the purpose of this paper this type of bituminous construction is called "Open-Graded Type" to be consistent with the nomenclature suggested by Mr. J. E. Buchanan at the last conference. He defined Open-Graded Aggregate as follows: "A graded aggregate with a limited proportion of particles passing No. 10 sieve and not more than 5 per cent passing No. 200 sieve."

Probably 95 per cent of the low cost surfaces constructed by the Construction Division are being placed on stabilized clay-gravel bases. This is desirable and essential in that it provides adequate base support for the bituminous surface and a smooth uniform surface upon which to mix and place the bituminous mat.

Both one course and two course bituminous wearing surfaces have been placed but recent experience has indicated that very satisfactory results can be obtained using one course construction for road-mix surfaces and for the most part such is the present procedure. The thickness of our mats are apparently materially thinner than those commonly laid in the Western States and range in thickness from approximately ¾ inch to 3 inches with the latter being the exceptional cases. The usual mat is less than 1¼ inches in thickness, although 1¼ inches is becoming more and more the standard thickness. It is recognized that although a final thickness approaching 3 inches is desirable, the thinner mat is being laid to serve traffic until such time as it is felt that ultimate stability of the base has been obtained at which time the final mat, usually plant-mix, can be laid.

In the case of the 1¼ inch road-mixed mat laid on a stabilized gravel base, the procedure and quantities are as follows:

Gravel aggregate in the amount of 125 pounds per square yard is placed on the road and mixed with from 0.65 to 0.95 gallon of MC-2 or MC-3 or cold application tar TC-5. Prior to the placing of the gravel the stabilized base is primed with from 0.1 to 0.3 gallon per square yard of MC-1 or TC-1. A frequent deviation from this procedure is to apply the prime coat following the mixing of the gravel aggregate and bituminous material and prior to its spreading. The seal coat may consist of a light fog seal of 0.1 gallon per square yard without cover, or a heavier seal coat with a sand cover.

With reference to the plant-mixed gravel surfaces, it can be briefly stated that the mixture consists of gravel conforming to the specifications given in Table 1 and from 4.4 to 4.8% by weight of cut-back asphalt MC-4 or MC-5, or an equal volume of hot application tar TH-3. Such mixtures are usually laid at the rate of 150 pounds per square yard. Present specifications provide for a seal coat of 0.2 to 0.3 gallon of bituminous material and 10 to 20 pounds of clean coarse sand. Both tars and asphalts are employed for sealing purposes.

The resultant mixtures obtained by either the road or plant-mixed method spread readily and compact under the roller satisfactorily. They are open-graded and unless sealed, exhibit a tendency to become hard and abrade. The mixtures are adequately stable and have shown no tendency to flush or become slippery. It appears that there is a comparatively wide range of bitumen content which can be used so that little difficulty is encountered in this respect. Less control is required by the laboratory than if a filler material and slow-curing oil were used. A more flexible mat would be obtained if slow-curing oil were used, but it is our experience that the above described procedures when used followed by a seal coat at the proper time, do result in the obtaining of satisfactory surfaces.

SECTION III

Construction of Bituminous Surfaces

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Review and Summary of Papers Prepared On Construction of Bituminous Surfaces

By

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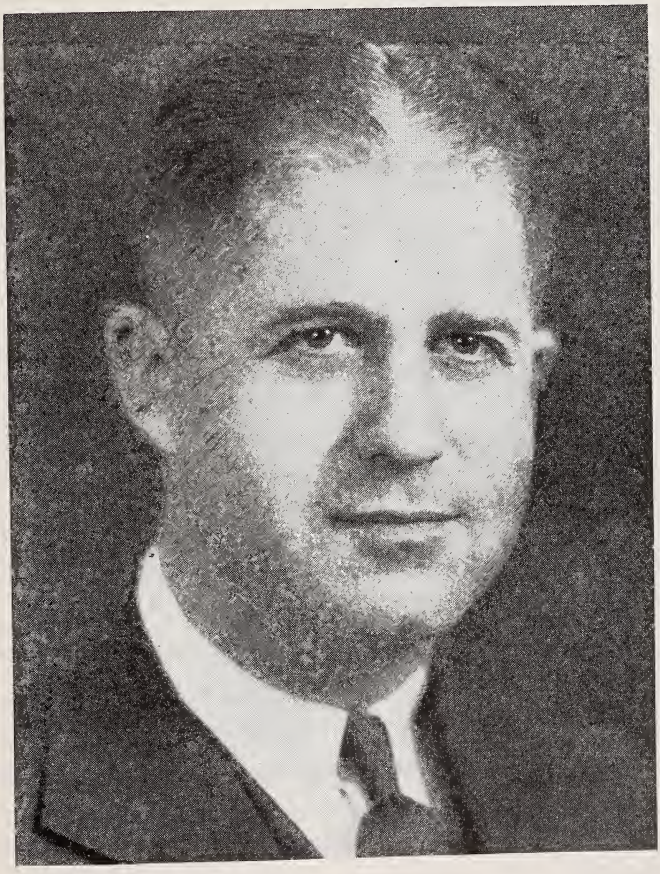


Mr. Chairman and gentlemen of the conference, at the outset I would ike to confess that this is my first year in attendance at your Montana neetings, and although I haven't been here before, the Michigan State Highway Department has been represented for the past three years, and we have always felt that we have been very well repaid and are very much pleased at the interest that is shown at the meetings and in this bituminous work.

On behalf of the Michigan delegation I should like to congratulate Mr. McKinnon and Mr. Cheney for the work they have done, and Mr.

Boggs of the Western Petroleum Refiners Association, and also the Mis sissippi Highway Department for the wonderful arrangements they hav made for this convention.

Even a casual glance at the preceding 3 volumes of the Proceeding of this conference reveals a great mass of information on both the theor and practice of bituminous construction.

Development in any field of engineering is necessarily slow, both i design and construction, so at first it might appear that unnecessary repeti tion may result from annual presentation of papers, particularly on con struction. This conference has, however evolved the rather unique pla of committee membership from widely varying geographical sections o the country where climatic conditions, soils, and materials and types o construction may thus be correspondingly varied. By rotating this autho membership annually, new developments and summaries of constructio activities may be presented yearly.

With the thought in mind that attempts by the authors to cove all types of bituminous construction may result in duplication of material we have digressed slightly from the evident plan of previous committee and have allowed considerable latitude in the selection, by the com mittee members, of the material to be presented in the individual papers You will note that while some of the papers cover construction details o several types of mixtures, others are concerned primarily with detailed descriptions of types with which the author has been most concerned.

The types of construction described in the papers submitted by thi section have been most varied, and in the conference which follows the report of the committee it is greatly to be desired that discussion wil cover not only the subject matter of the papers herein abstracted, but related types as well.

The limited time available has prevented a circulation of the papers among the committee members, therefore no attempt has been made to present the views and attitude of the members as a whole.

Twelve papers have been submitted to this section, the titles and authors being as follows:

"Sand Bituminous Low Cost Road Construction in Florida" by John R. Slade, Construction Engineer, State Road Department, Florida.

"Bituminous Surfaces for Airport Runways" by W. R. Macatee, District Engineer, The Asphalt Institute, Cincinnati, Ohio.

"Mississippi's Experience with Loss in Penetration" by G. S. Covert, Assistant Construction Engineer, State Highway Department, Mississippi.

"Record of Construction of Black Base Asphalt Macadam, Penetration Type" by Carl R. Reid, Engineer of Materials, Oklahoma State Highway Commission.

"Present Practice in Bituminous Road Construction in Alabama" by Leon Gottlieb, Bituminous Engineer, State Highway Department, Alabama.

"Developments in Construction of Bituminous Surfaces in Wisconsin" by A. T. Bleck, Construction Engineer, State Highway Commission, Wisconsin.

"Liquid Bituminous Mix Using a Traveling Plant" by H. G. Groves, Bituminous Engineer, Yale Oil Corporation, Billings, Montana.

"Plant Mix Bituminous Surface Construction in Nevada" by Huston D. Mills, Assistant State Highway Engineer, Nevada State Highway Department.

"Medium Cost Mixed in Place Bituminous Surfaces" by E. N. Blackwood, State Construction Engineer, State Road Commission, West Virginia.

"Surface Treatment" by J. F. Pierson, Construction Engineer, State Highway Commission, Baton Rouge, Louisiana.

"Stabilized Gravel Base and Bituminous Surfacing in North Dakota" by H. J. Donovan, Division Engineer, North Dakota State Highway Department.

"Gravel Stabilization with Michigan Slow-Curing Oils" by J. D. Brown, Engineer of Bituminous Research, Michigan State Highway Department.

FLORIDA

We are pleased to have the latest developments in sand bituminous ow-cost roads in Florida, accounts of which have been given in the ast. It is always interesting and instructive to hear from all attempts o use aggregate, which while they may not conform to accepted standards f ideal composition, have the big advantage of low cost, and in the resent example are already on the road. Such types of roads have made ossible the construction of many hundreds of miles of excellent surfaces vhere the cost of imported materials would markedly limit the con-truction.

Present construction of this type of pavement utilizes road ma-erial of sand, or its mixtures with gravel, stone or shell, including ad-nixtures of other material, if necessary, to correct deficiencies in grading.

Mr. Slade states that good drainage is essential to the success of hese pavements, and particular attention is given to the bearing value f the subgrade, which should be a minimum of 25 lb. per sq. in. The uthor describes the test in detail.

The State Road Department of Florida has used successfully cut-back asphalt of a grade similar to RC-1, a coal tar base containing pitch il and fluxed with water gas tar or its distillate, and petroleum tar having some of the characteristics of coal tar.

For a sand-bituminous road mix pavement of 6-inch thickness the existing road material is plowed to a depth of 10 inches, dragged with spring tooth harrows to remove roots and other foreign material, and if necessary additional aggregate is added to correct deficiencies. The road is then bladed to a uniform grade and cross-section.

The amount of bituminous material is determined by laboratory i vestigation and may be modified by visual inspection based on past e perience.

Application of bituminous material is at the rate of 0.35 gal. p sq. yd. per application, each one of which is followed by continuou mixing with harrows. After 2 gal. per sq. yd. has been applied, th entire width of pavement is plowed to obtain uniform depth and mixin with harrows is continued. The material is then windrowed to a dept of 3 in. with a 12-ft. grader, from center to outside. Application of bitum nous material, harrowing, and blading is continued until a homogeneou mixture is obtained. In mixing the upper 3 in., harrows are used and durin the last three or four applications of bituminous material the retrea mixing machine is used continuously.

After mixing, the surface is bladed to the desired grade and cross section, and when partially set up it is then rolled. After setting severa days more the pavement is given a final longitudinal and diagonal rolling

A seal coat is not usually required with cut-back asphalt, but wit coal tar and petroleum tar a seal at the rate of 0.2 to 0.35 gal. per sq yd. is applied about a month after mixing operations are completed.

It is reported that the cost of this type pavement has averaged 6 cents per sq. yd., using an average of 5 gal. of bituminous material.

Investigations have been made relative to the use of a mixture o powdered hard asphalt and slow-curing oil and in the future will be use as an alternate to the three other materials. It is also planned to allov the use of the traveling plant as an alternate to road mixing.

MISSISSIPPI

The information presented by Mr. Covert has added to the rapidly accumulating data on the characteristics of asphalt recovered from pave ments.

Although as Mr. Covert states, Mississippi's experience with hot mix has in the past been limited, it is indeed gratifying to note how carefully the project under discussion has been observed since construction, and the correlation of cracking with subgrade conditions.

This project consisted of 20 miles of 4½ inch pavement; 3 inches of asphaltic concrete base course laid over 8 inches of compacted sand clay gravel base, and 1½ inches of modified Topeka wearing surface.

Within 2 or 3 years after completion there was excessive cracking of the wearing surface over sections of bad soil, with no apparent de formation in cross-section.

After five years the cracking had progressed into the base course, and the penetration of the asphalt had dropped from an original of 45 down to 24 (Abson recovery), a loss of 44% in this period. No data was given on the penetration of the asphalt immediately after construc tion.

Mr. Covert's conclusions that "part of this cracking is due to the so-called 'drying out of the mix,' rather than to the hardening of the

sphalt'' emphasizes a fact that should not be lost sight of in the attempt correlate penetration and ductility values with service behavior. The mount of asphalt and mineral filler present in a mixture and the mineral radation, and the density of traffic to which it is subjected, must be refully considered along with the characteristics of the bitumen.

Michigan has a number of examples of pavements, the condition of hich can be explained only by the proper interpretation of the above ariables. In one recent instance a badly cracked sheet asphalt pavement howed penetration and ductility values on the recovered asphalt which ormally indicates a satisfactory pavement. Its present condition can nly be explained by the relatively low bitumen and high filler contents. t is apparent that the pavement, although of high stability, possesses insufficient flexibility (or ''ductility'' as a mass) to withstand the tensile orces set up in contraction or the slight movements due to the unstable subrade or base conditions.

OKLAHOMA

Mr. Reid, of Oklahoma, describes the construction of an asphalt macdam base, 6 inches compacted thickness and 24 feet wide, on U. S. Highway 277.

The soils survey showed no possibility of high-ground-water and o need for extra thickness of base.

Traffic conditions, requiring a higher type base course than soil-ound gravel or stone, and a nearby location of commercial limestone rompted the department to construct a two course asphalt macadam ase, which, during its economical life, is to serve as wearing surface. When necessary this base will be covered by a dense graded bituminous nixture.

Due to the lack of experience with this type of construction on the art of the engineers, the specifications adopted were those in general se. Soon after the production of base stone was begun it was apparent hat far too little stone of required specification grading could be produced to permit the continuous operations necessary for good work, or to complete the project in the normal construction season.

It was finally decided to construct the remainder of the project as a test section to determine whether a somewhat smaller sized aggregate would allow the penetration of a sufficient quantity of asphalt to insure complete coverage of the aggregate. The principal changes in aggregate grading were the increase in the total passing the 1½ inch square screen from 5-25% up to 30-60%, and that passing the ¾ inch from 0-5% up to 0-15%.

It was found that considerably less asphalt was required with the smaller sized aggregate, and soundings made later showed that the voids were fairly well filled. An average of 0.6 gallon of asphalt per square yard per inch thickness was used. It is of interest to note that within 30 days the original penetrations of from 86 to 97 had dropped to an average of 59.

The author states that the riding qualities of the road are very unsatisfactory and lists the following possible causes:

1. Insufficient "keying" of the base rock.
2. Incomplete rolling.
3. Too great or uneven application of key rock.
4. Rolling of key rock before asphalt had cooled sufficiently.

We trust that other engineers thoroughly familiar with this type of construction may have some suggestion to offer during the discussion which follow this session.

ALABAMA

Based on the experience gained in the construction of over 1,000 miles of surfacing over a period of years, Mr. Gottlieb reviews present practice of bituminous construction in Alabama.

The author stresses the importance of complete soil studies preliminary to construction operations, the object being to decide the proper amount and type of materials to be used for base course. Materials are selected which have low volume changes, sufficient adhesion and stability to carry the traffic to which the road will be subjected.

Over plastic soils, a minimum of 1 foot of sandy loam is used as a subgrade topping before placing a base course of sand-clay 8 inches minimum thickness. Over friable soil, a 6-inch gravel base has proven satisfactory.

Mr. Gottlieb mentions the economy and efficiency which has resulted from the experience gained by the engineers and inspection department personnel as well as the contractors. A reduction in unit costs of nearly 40 per cent has resulted.

Base course construction is placed in two layers, mixing and shaping operations being carried out by motor patrols, plows, and harrows.

Bituminous treatment begins with the application of prime coat at the rate of 0.3 gal. per sq. yd. This year the bituminous material for hot application has been reduced from a minimum of 0.4 gal. to 0.35 gal. and the aggregate reduced from ½ cu. ft. of 1 inch to ½ inch material to 0.45 cu. ft. of ¾ inch to ⅜ inch sizes. The use of the smaller aggregate has resulted in a reduction in cost of 1½ cents per sq. yd. and produces a more uniformly satisfactory surface. Bituminous material is usually 150 to 200 penetration asphalt although RT-12 is sometimes used.

Present practice is to finish the surface treatment with an application of 35 lb. per sq. yd. of cold plant mix material. This provides a very uniform and densely graded wearing surface. The plant mix seal specifications are as follows:

Passing ½ inch sieve	100%
Passing ⅜ inch sieve	85-100%
Passing No. 4 sieve	40- 90%
Passing No. 10 sieve	20- 65%
Passing No. 40 sieve	5- 35%
Passing No. 200 sieve	0- 10%
Bitumen and liquefier	5- 8%

When the mixture is to be shipped, an asphalt of 85-100 penetration plus a liquefier is used but when the mixture is made on the project 150-200 penetration asphalt is used without a liquefier.

WISCONSIN

Mr. Bleck traces the development of secondary road bituminous surfaces in Wisconsin starting in 1937 when all untreated gravel and earth roads were given applications of oil to dustproof and conserve the surface. Pursuing this policy all new construction, including detours, on which the use of gravel or crushed stone is contemplated, receive a similar treatment, further mat construction being deferred at least a year. During this period any necessary corrections can be made to the subgrade.

In the majority of cases new mat construction is road mix using SC oil, over 4 to 6 inches of stable base, although some plant mix has been used. Mr. Bleck mentions the necessity of reducing the maximum size of aggregate to 1 inch, since some segregation occurs with finishing machines. The thickness of the mats at the present time are 3 inches, placed in two courses, each course correcting minor irregularities in the course or base below. The author is inclined to believe that, based on the results obtained with the lower course, a single layer from 2 to 2½ inches thick is sufficient.

Mr. Bleck describes in detail an instrument for the determination of bitumen based on the principle that light passing through a colored solution loses intensity in porportion to the strength of the solution. To those interested in a relatively new departure in methods of bitumen determination a study of the original article is recommended.

We are wondering if the author has investigated the relative color intensities of different types of the same grades of oil, such as some straight run residual oils, gas oil cut-backs, and highly cracked oils. It is possible that a definite color intensity may not represent the same bitumen percertage for the various oils.

It is our thought that perhaps even the simplified form of this apparatus may be too complicated for accurate results in the hands of the average inspector. Michigan uses the rugged hand powered Rotarex, which is included in all bituminous inspectors field cases. Oil mixes are quickly analyzed, although for road mixes a moisture determination on the aggregate just before mixing with oil would be required.

NORTH DAKOTA

Mr. Groves describes an interesting construction project by the North Dakota State Highway Department, the unusual feature of which consisted of a traveling plant mix utilizing a slow-curing oil of much higher viscosity than is customary in cold mix operations.

The bituminous mat was constructed over a 4-inch stabilized gravel base to which a prime of SC-2 oil was applied. The windrow of gravel and filler, manipulated when necessary to reduce the moisture content below the required minimum of 2 per cent, was then ready for the traveling plant.

Satisfactory construction with the viscous oil used undoubtedly demands favorable weather conditions since the author observes that aggregate temperatures were as high as 108° F. and after oil had been added at 300° F. the completed mix temperature was 118° F. While the average temperature was no doubt much lower, still the mention of the fact that it was sometimes impossible to continue all night work points to the conclusion that warm weather is necessary to this construction. It is interesting to note that motor patrols successfully maintained such a mixture.

Of interest to note is base construction a full 5 feet wider than the bituminous mat. This provides stable shoulders which will carry the traffic and will be ideal for surface or other treatment should the demand require further widening of the road.

Another departure from average construction practice was the one foot widening on the inside of all 2 to 5 deg. curves and increased thickening to 2½ inches. To our mind this is good practice since it relieves maintenance work and provides a uniform surface so often unsightly when built in later with dissimilar materials.

Of further interest was the immediate application of a seal coat when the normal surface would undoubtedly wear for a number of years. We are wondering if the somewhat non-uniform appearance of the average seal coat does not detract from the otherwise uniform oil mix surface, although it is possible that blading so stiff a mix may produce sufficient segregation to call for a seal.

NEVADA

Mr. Mills, in discussing bituminous construction in Nevada, enumerates the changes made since 1932 in crushed stone plant mixes using both slow-curing and medium curing asphaltic oils. No definite preference has been indicated as to the bituminous materials.

Among the improvements noted have been the separation of aggregate into three instead of two sizes in bins, presumably to decrease segregation and insure uniformly graded mixtures. The aggregate passing the No. 10 sieve has been reduced from an allowable maximum of 60 per cent to 45 per cent, which should increase the non-skid properties.

We are particularly interested in Mr. Mill's comments on the relative merits of the riding qualities of surfaces produced by mechanical spreading and finishing machines and blades and his expressed preference for the latter. Our experience in Michigan with oil-aggregate has shown that black-top pavers (as distinguished from finishing machines riding on side forms) produce satisfactory riding surfaces, especially when laid in two layers. This may be attributed partly to the use of the relatively fluid SC-6 oil (proposed A.A.S.H.O. specifications) and partly to the fact that an appreciable percentage of the aggregate is uncrushed, the two conditions resulting in a most workable condition until full compaction is attained.

Nevada's crushed stone mix probably most parallels our bituminous concrete mixes as concerns workability and for this work, as well as

sheet asphalt construction, we insist on the use of finishers on forms true to grade. It is further stipulated that plant production synchronize with finisher speed to reduce interruptions to a minimum. While it is true that blades will do an excellent job on materials similar to our oil aggregate, we aim for the lower costs obtained with large plant output.

NORTH DAKOTA

Mr. Donovan gives a history of the construction of a stabilized gravel base and bituminous surfacing on a North Dakota project.

Specifications for the mixture of gravel and soil binder required a P. I. on the fraction passing a No. 40 sieve between 8 and 15, with the sum of material passing the No. 40 sieve and the P. I. not exceeding 50, and with the further requirement that the liquid limit of the same fraction not exceed 1.6 times the P. I. plus 14.

The grading of the combined material conformed to the following requirements:

Passing 1 inch square sieve	100%
Passing ¾ inch square sieve	85-100%
Passing ⅜ inch square sieve	65-100%
Passing No. 4 sieve	55- 85%
Passing No. 10 sieve	40- 65%
Passing No. 40 sieve	25- 50%

The combined aggregates were hauled to the road in two equal courses at the rate of 1275 cu. yd. per mile. The first, laid late in the fall of 1937, required reworking in the spring of 1938 when work was resumed. The second course was bladed back and forth to insure proper dispersion of the clay, since the traveling plant alone did not accomplish this end. The plant, however, did an excellent job of mixing with water. After laying to section, more water was added and compaction was obtained with pneumatic rollers.

Base construction was completed with 0.5 gal. per sq. yd. application of SC-2 as a tack coat.

The author expressed the opinion that a stabilized gravel job should be open to traffic during construction, which is in accord with experience in Michigan.

Bituminous Surfacing

The combined pit run aggregate and natural soil filler specification requirements for the bituminous surface were as follows:

Passing ¾ inch square sieve	100%
Passing ¼ inch sieve	50- 75%
Passing No. 10 sieve	35- 65%
Passing No. 200 sieve	5- 14%

The gravel was dumped into spreader boxes to give a windrow 8 feet wide and 6 inches thick. The percentage of 200-mesh material varied between 5 and 7.5%, and the amount retained on the ¼ inch screen was between 25% and 33%. Before mixing with bitumen the aggregate was bladed to reduce the moisture content to a maximum of 2 per cent. It

was found that this operation nullified the purpose of the tack coat by depositing a film of the fines over the entire base. An additional prime application was found necessary.

Considerable difficulty was experienced, especially so at the start, in mixing the SC-7 oil which is much more viscous than is usual for cold mixes. There was an unusually large amount of uncoated particles which raveled out and were thrown out to the shoulders by traffic. It was almost impossible to remove traffic ruts made after the initial compression. The general conclusion seems to be that SC-7 oil is too viscous for satisfactory workmanship in cold mixing.

The surface was completed with a double seal consisting of 0.15 gal. per sq. yd. of RC-2 covered with coarse sand at the rate of 60 tons per mile, followed by a second application of RC-2 at 0.30 gal. per sq. yd. covered with sand at the rate of 126 tons per mile.

MICHIGAN

The paper by J. D. Brown, of the Michigan State Highway Department, presents some interesting results of a preliminary investigation of gravel base stabilization with slow-curing residual oils. The study was aimed particularly at determining the source and extent of any detrimental effects of excessive quantities of moisture in the aggregate at the time of mixing and placing in the road. The absorption characteristics of the mixture were used as a measure of its performance on the assumption that the capillary properties were the major sources of deterioration in such mixtures under practical conditions.

The results of this investigation should be viewed against the background of general experience with moisture in bituminous mixtures which until quite recently has regarded moisture in any more than negligible quantities as definitely detrimental. The statement has been made after comprehensive surveys that bituminous mixtures in which aggregate contained more than 1½ or 2 per cent moisture at the time of placement resulted in unsatisfactory service behavior. On the other hand, more recent developments in soil stabilization with low percentages of bituminous materials had indicated that a certain amount of moisture is essential to the uniform distribution of bitumen throughout the mass as a necessary aid in mixing and placing.

Mr. Brown's paper may serve to clarify these apparently contradictory observations and appear to suggest the specific function of moisture in mixing and the necessary procedure to accomplish this function and avoid its detrimental effect. Briefly stated, the laboratory tests appear to demonstrate that a certain amount of moisture followed by a drying period before placing the mixture in the road is required to accomplish an effective distribution of bitumen and eliminates the detrimental features. The moisture serves as a distributing agent for the bitumen and the drying period brings into play capillary movement which is essential to depositing the bitumen where it will seal the mixture. Even with excessive moisture the drying period apparently eliminates bad results. To what extent the drying period may be postponed until the mixture is placed seems to depend on the viscosity of the residual oil and whether or not the road is subjected to traffic immediately.

It should be pointed out that the investigation deals only with behavior of the mixtures during and immediately after construction. Within these comparatively short periods of time the mixtures showed very favorable resistance to absorption. Whether or not displacement of the bitumen or stripping by water may still occur over longer periods of time is another problem and in any event is shown to be independent of the initial moisture content of the mixture.

THE ASPHALT INSTITUTE

Mr. Macatee reviews present practices and trends in airport surfaces. Of particular interest to the highway engineer is the movement on foot to develop more or less emergency landing fields, designated as "Flight Strips," adjacent to, and a part of the highway right of way, which automatically becomes the responsibility of the various highway departments.

The most pronounced trend in airport surfaces seems to be toward lighter and less expensive surfaces. Crushed stone bases are increasingly replacing the more expensive rigid bases.

Rational design based on the bearing capacity of the soil is replacing rule-of-thumb methods and in many cases results in considerable economy. Soil stabilization utilizing existing soil, with admixtures when necessary for increased mechanical stability, not only furnishes the necessary water repellent qualities but greatly increases the stability of the base. Not only clay soils, but sand and gravel mixtures are being stabilized with a wide variety of bituminous products such as slow-curing, medium-curing, and rapid-curing asphaltic oils, and asphaltic emulsions.

Sub-bases are being constructed of penetration macadam and asphaltic concrete with thin rich carpet coats of soft asphalts.

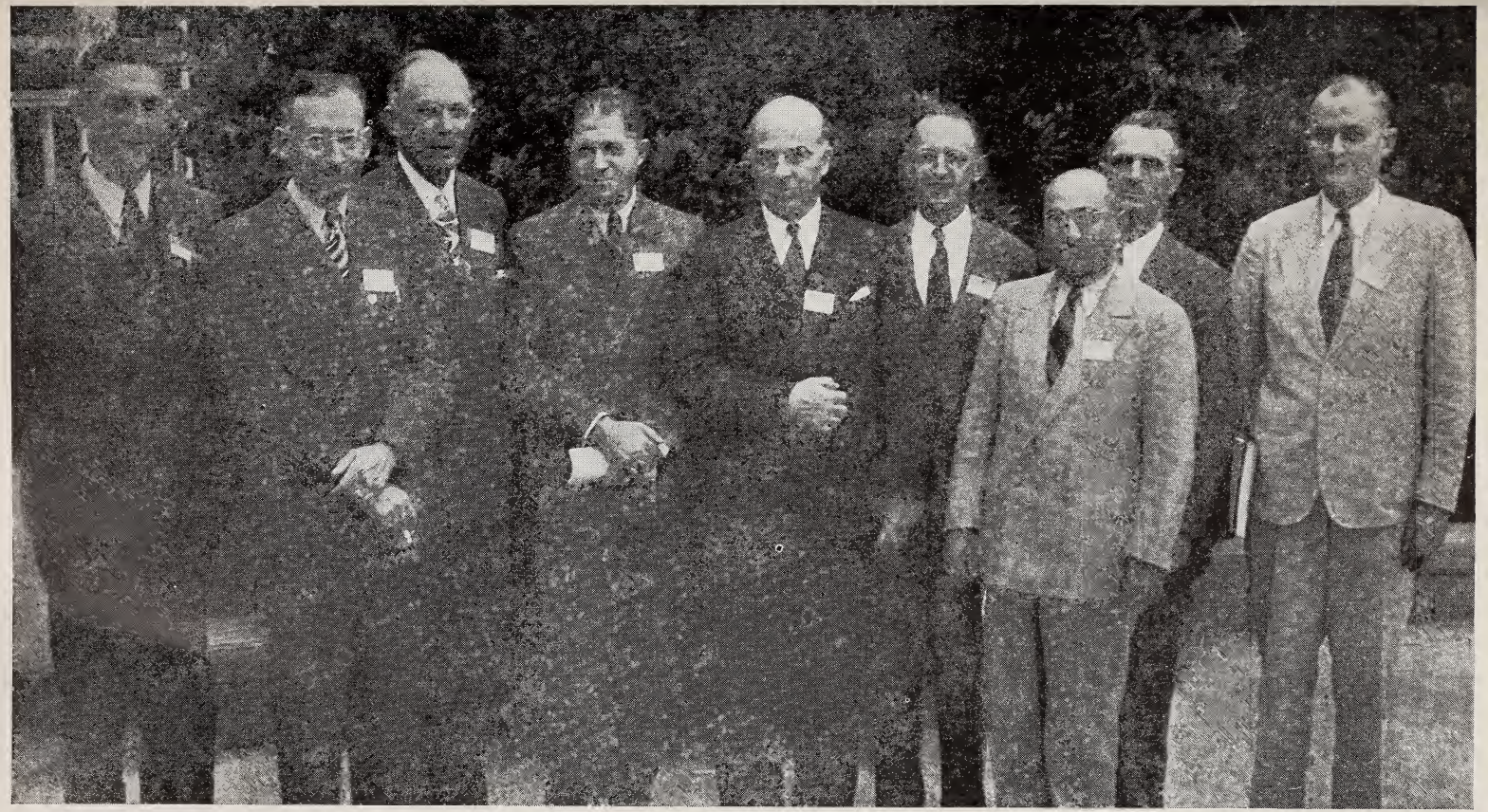
There is a pronounced trend towards the use of asphalt mixtures for producing cut-off or side walls at the sides and ends of run-ways to prevent water from adjacent, unpaved areas from finding its way horizontally under the run-way surface structure. Coarse stone, in one airport construction, was lightly coated with bitumen for the necessary binding qualities. This allowed the complete filtration of surface water to clay tile drains below.

In addition to the conventional type of run-ways are the landing mats to which the run-ways are appendages. These mats are especially valuable in the fast developing blind landing which aviation is perfecting. The Cleveland airport, one of the prominent American airports, utilized a 5″ slag base with an intermediate 2″ course of penetration macadam covered with 1½″ of cold-laid plant mix asphaltic concrete.

Mr. Macatee summarizes the most pronounced trends in the construction of airport surfaces as follows:

1. Toward lighter, less expensive forms of surfacing.
2. Toward utilizing local materials more extensively.
3. Toward more extensive use of flexible forms of surfaces and bases

4. Toward thicker films of asphalt surrounding aggregate particles.
5. Toward the use of softer asphalts.
6. Toward greater use of liquid asphalts such as MC and RC groups.
7. Toward more extensive use of asphalt stabilization of bases.
8. Toward constructing cut-off walls of asphalt mixtures at ends and sides of runways.
9. Toward having at hand fully developed and quickly available plans for future extensions of runways and other airport facilities.



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Conference on Construction of Bituminous Surfaces

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Mr. Steinbaugh: Is there anyone here who would like to start the discussion on any part of this subject?

V. G. Gould: Iowa

Is it in order for me to start the discussion with a question?

There is one subject which is covered in these papers which I am sure is of general interest to all highway departments, and I have hastily jotted down three questions with comments as affecting conditions in our own state intended to be of an explanatory nature. I present these three questions which the chairman may either eliminate or ask, as he sees fit according to the amount of time available.

The first is with regard to compaction of stabilized bases. In Iowa we use a 5" thickness for the soil-aggregate, bituminous or rolled stone type, with densities specified as 100% Proctor in all cases. We feel that if 100% Proctor densities are not used thicker bases would be necessary and base edge failures would be common. The method of compaction is not specified. The question, then, is "Do other states specify densities, and if so, what densities are specified?"

The second question has to do with bituminous stabilizers. Last year, in our state, we used an SC-1 stabilizer for soil aggregate base construction. This year the base is failing generally due to rutting, corrugating and cracking. Investigation indicates that the SC-1 is being replaced with water, causing the unstable condition. Question: "Has any other state found SC's of like viscosity to be readily displaced by water, and has there been any conclusive evidence to show that positive SC's are less readily displaced than negative SC's?"

The third question is in regard to mixing methods. This year, in our state, we require batch plant mixers for soil-aggregate type bases. Question: Do other states use plant mixes and do they consider plant mixes versus road mixes are justified?

Mr. Steinbaugh: Thank you, Mr. Gould. I think this subject of compaction of sub-grades is a very good one. I would like to ask if anybody here representing any other state highway department has a factor of compaction by which you measure the amount of compaction in your subgrade, or do you just hope that you get it completely compacted? Is there anyone actually measuring that, except Iowa?

I. E. Russell: Wyoming

In our work we make our preliminary soil surveys and then, during actual construction, we make check tests in the field, and the same material that is taken in the field is sent to the laboratory again for recheck on the standard method of compaction to find out whether or not the actual work is being done in such a way as to

get the compaction that we want. Of course we are shooting for 100%, but we don't think that it is advisable to carry moisture contents up as high as 15 and 20% in order to get just another pound or two of density. In other words, we might have 90% or 95% with 8% of moisture in the field, yet our curve is so flat that we would have to increase the moisture content too much for any appreciable increase in density.

Mr. Steinbaugh: Do you construct your bases under traffic, in general?

. E. Russell: Wyoming I am speaking of subgrades. That was the question, wasn't it?

Mr. Steinbaugh: I understood it was bases for bituminous surfacing, not entirely subgrades.

. E. Russell: Wyoming All of our base courses are watered and thoroughly rolled and mixed with soil binders if necessary. In order to determine whether or not we need soil binders in order to set up our base courses, we use cementing value tests on the -10 material. Where the base course is to be immediately followed with bituminous mats we hold that cementing value to a minimum.

Mr. Steinbaugh: Do you have any way of telling when your base course, for instance, is compacted to the point of where you can consider it is ready for your bituminous surface?

. E. Russell: Wyoming Not on the bases, no.

Mr. Steinbaugh: Thank you, Mr. Russell. Has anyone else anything to say on this subject? Do any of these soils experts around here have anything to say on that? It seems to me that where bases are to be followed immediately by a bituminous surface, some method of determining the compaction is quite a reasonable thing to do, particularly, as has been pointed out, for compaction of the edges, where if the compaction is not complete, failure will result at some later date. But I know that in our case we try to arrive at that, by building the bases a year or so ahead of the time we are going to put the bituminous surface on, and let nature do it. Where it is to be followed immediately by a bituminous surface, I imagine that some form of testing or some way of reassuring yourself would be a desirable thing.

. H. Swanberg: Minnesota We make density determinations of our stabilized gravel and we have found some variation in that. We have given some consideration to including in our specifications a density requirement, but one point in that connection that concerns us is that because of variations in the specific gravity of our aggregate materials we wonder whether that density should not be specified on the basis of void content rather than of pounds per square yard of depth.

A. E. Stoddard: Iowa

Our specification is based on the determination with th actual materials you are going to use, under Procto density, not necessarily in terms of so many pound per cubic foot.

Mr. Steinbaugh: You take the quality of the material into consideration

A. E. Stoddard: Iowa

Yes. There is a question in that connection that I wan to ask, in regard to your traffic bound bases. Have yo made any determination of the density you get b that method?

J. G. Schaub: Michigan

In Michigan we don't make any determination of density Our problem is probably a little bit different from mos states. We have quite a little tourist traffic in Michiga and have to build most of our roads under traffic Consequently, we keep the moisture content of soil ag gregate mixtures slightly low, in order to provide free dom to travel over it as we lay it.

As Mr. Steinbaugh says, we depend on roller com paction and let the bases lie a year to let further traffi and frost action do what they will toward packing.

Mr. Steinbaugh: Professor Housel, have you any comment to make on this

Prof. W. S. Housel Michigan

I might amplify it just a little. We have used the ful Proctor control on several experimental projects an since that time, where the bases are traffic bound, w have spot checked different projects for density, an find that we are getting densities in well controlle gradations, which compare very favorably with the Procto densities we had on the experimental projects. Howeve it isn't a routine part of every one of the projects t run the Proctor test.

Mr. Steinbaugh: Does anyone else have any comment?

The next question that was asked is, "Has any othe state found SC oils of light viscosity to be readily displace by water, and is there any conclusive evidence to sho that positive spot SC's are less subject to displacemen than negative spot SC's?" That is a subject that shoul start an argument. How about the first part of it, th question is, "Have you had any trouble with your slow curing oils, particularly the light oils, SC-1 and SC-2, tha later were displaced by water, causing failure in tha way?

Has anyone had any experience with them being satis factory, where they have not been giving any trouble?

F. V. Reagel: Missouri

We have had both results with a little heavier SC. B that I mean, where we had gravel as the major phas we had trouble with elimination, but where we use mixtures of soils and granular material, using an SC- or an SC-3, we have not noticed that effect.

Mr. Steinbaugh: I believe in our case we have found that the addition of a small amount of limestone dust has quite appreciably helped that situation, even though there were sufficient fines in the mixture, normally, of sand or other materials. A small amount of limestone dust seemed to have quite a beneficial effect.

Have you any comment on that, Mr. Schaub?

. G. Schaub: Michigan

That is particularly true in our oil aggregate mixtures, in which we use a slightly heavier viscosity oil.

Due to the variable properties of our aggregates in Michigan, we submit all the fines, or the finer portions of the aggregate, to the water-asphalt perferential test. We find our silica aggregates tend to strip and show a preference for water instead of asphalt, so, as Mr. Steinbaugh says, by adding limestone dust, or marl or beet sugar lime, or calcareous material of that kind, we sort of give it a property in the reverse, so that it has a preference for the oil rather than the water.

Referring to Mr. Brown's paper, we conducted the experiment, using slow-curing oil of light viscosity to make the mixture water repellent in base work. It was undertaken for two reasons; one, to reduce capillarity and keep subgrade moisture from our bituminous surfaces, and also to find an outlet for the producers of Michigan oils. They don't produce asphalts but do produce slow-curing oils and road oils; so we tried out this slow-curing oil for making the mixture water repellant. It is a little too early to tell whether we are going to have any stripping action, but we are rather hopeful it will not be serious. We are interested in the experience other states have had.

Mr. Steinbaugh: Mr. Reagel, have you anything to say in regard to the last part of this question, in regard to whether positive or negative spot oils show any difference in the results?

F. V. Reagel: Missouri

I might say the oils I just mentioned were all positive spot materials. We have had little experience, if any, with a negative spot SC material. It so happens that the material marketed there most economically is the cracked material, and I believe there is some evidence available, however, in laboratory tests, and perhaps Dr. Winterkorn could give it to you better than I could, but there seems to be an indication that the cracked materials do give better results than the uncracked. That is determined by absorption and slacking tests on specimens in the laboratory.

Mr. Steinbaugh: Is there any further comment on the question of using cracked or uncracked oils for either primes or in mixtures, and the relative displacement by water?

Dr. Hans F. Winterkorn: Missouri

I may say something about the experiment to which Mr. Reagel referred. I referred to it this morning. I just want to say that mixtures of SC oils with different soils were studied, and we had some SC oils which were non-cracked, some which were cracked, and one sample from California; we find that the California oil is in between with its adhesive properties. So on one hand you have the Mexican type, on the other hand the cracked type, and the intermediate is the California oil.

Mr. Steinbaugh: Is there any other comment on that subject?

H. G. Nevitt: Missouri

I just want to make one remark. When they used an SC-1 oil they cut down the effective asphalt content, giving the effect of a lean mix, and speeded up the oxidation. We have noted this several times. If you hold the bituminous content of a mixture to the prescribed minimum, then use an SC-1 oil, (you are really cutting down the asphalt content), it might account for the trouble noted in that case.

V. G. Gould: Iowa

There is just one comment I might add with regard to what Mr. Reagel said about the addition of aggregates. In this particular case the surfacing material on the road was incorporated with the soil so that we estimated (not accurately) that there was about 70% aggregate and 30% soil of that mix.

Mr. Steinbaugh: The third question: "Do other states use plant mixes and do they consider that plant mixes versus road mixes are justified?" Perhaps we should clarify that question a little bit. Are we talking now, about batch plant mix, and for hot or cold bituminous mixtures,

V. G. Gould: Iowa

Just for soil aggregate stabilization.

Mr. Steinbaugh: What is the experience in regard to that? Is every state doing its soil stabilization by mixed in place methods, or is anyone using plant mix? That is for soil aggregate mixture.

J. H. Swanberg: Minnesota

Most of our work has been road mixing, but this last season we have had a number of projects in which we used a traveling plant, and one project this year in which we used a central plant, although in each case they were continuous mixers. We are inclined to believe that the use of either a traveling plant or a stationary plant is preferable to road mixing. Particularly is that true where you are using heavy clay binders.

We are also inclined to believe that if it were possible to use a lighter clay binder rather than a heavier

clay binder, that is preferable because of the inability in some cases where we have considerable moisture by rainfall, to pulverize or break down the heavy clay binder. Even in the plant on the one job where we used the stationary state-owned plant we found that the clay binder was not as finely pulverized as it might have been, and that has given us some trouble, such as concentration of moisture and instability of our stabilized gravel mix, which in some cases was quite serious because of inability to break down that heavy binder.

Mr. Steinbaugh: Thank you.

I might add that Michigan a year ago started using plant stabilized gravel entirely for both maintenance and construction of bases. We have been doing it only a year; there is no doubt but that it costs considerably more to start with, because of the investment in plants that is being charged off, but in certain parts of our construction we would not consider going back to the old system. Where we are applying a resurfacing, of two or three inches of gravel over an old gravel road, particularly where it is under traffic more or less of the time the plant stabilized gravel is far more satisfactory than the old system of trying to stabilize it on the road. It is also, of course, much more uniform, and we get away from the discomfort to the traveling public in trying to stabilize it and having that soupy mess on the road. Also, I find that the gravel which is pre-stabilized with the proper amounts of clay and chloride, and then laid on the road, doesn't tend to get as slippery under wet weather conditions as the road that is stabilized by mixed-in-place methods, evidently for the reason that the clay is better distributed and the control is much better.

Whether we are going to continue this entirely for all of our new base construction we haven't yet made up our minds. The prices of stabilized gravel are gradually dropping as the contractors find out just what is involved in the manufacture of the material. We are laying our bases in layers of stabilized material. Where we place a bituminous surface in the same year, I think it is quite desirable that we use plant stabilized gravel. Whether it is desirable in all cases is going to be a question of economics, but without a doubt we will not return to the entire use of non-stabilized material for all purposes. At least fifty per cent of our gravel will be stabilized.

Is there anyone else who has had experience on that use of plant stabilized gravels? How about Indiana? Is Indiana represented here?

E. B. Lockridge: Indiana

I don't know whether our materials engineer is in this meeting or not. I think he can give a better explanation of that than I can. We have had some experience in an effort to stabilize both mixed on the road and plant mix. We have used both gravel and stone. I am not so familiar with all the details of this process because I have just recently come into the Construction Division, having formerly been with the maintenance work.

We finished a stone stabilization project this summer where material was made from a plant mix and laid using forms, and it just looked awfully good as work was finished. We haven't had it through the winter and I don't know what will be the result of this test after the elements have had a chance to work on it. It was found in this case by the contractor that he could lay plant mix stabilized material using forms and finishing machine (optimum moisture content being permitted) cheaper than if he had tried to lay it without the use of forms and shape it up with blade and otherwise later. We got a very nice looking job when it was completed.

Mr. Steinbaugh: Thank you.

Is there anyone else who has had stabilized gravel for either maintenance or construction purposes?

T. R. Perry: Iowa

I would be interested to know what type of equipment you are using in Michigan to secure your mixes. I might say that in Iowa the contractors are using the old 27-E concrete mixers, and a few of the twin pug mill type.

Mr. Steinbaugh: We are using pug mill entirely for the mixture of our aggregates, usually a Pioneer or some similar type of plant, with clay feeder and shredder and a device for chloride being added at the same time.

Another point that hasn't been brought out, we have had under certain conditions, particularly in resurfacing, some very good results in the use of some type of a black top paver for the laying of 2″ or 3″ courses. We used two or three types of machines of that kind and got very good results from them, although not in the heavier thicknesses such as six or eight inches.

Does anyone else have any comment?

J. W. Powers: Arizona

May I go back to your second question a moment? A gentleman asked about stripping. In connection with certain work that we have done on swell test, which is an indication of stripping, we find that those oils that have positive spot will strip or swell less readily than those that are negative, and the higher the xylene equivalent, up to 100% xylene equivalent, the more you can absolutely diminish the swell with that particular

type of material, and as you go into the negative material the swell increases very rapidly.

Mr. Steinbaugh: Has anyone else any comment on that subject again? I have heard the opposite view expressed. Have you anything on that, Professor Housel?

Prof. W. S. Housel: Michigan

I have nothing to say on that particular point in connection with positive spot tests. There is one comment that might be made in connection with the work reported by Mr. Brown. There have been several comments before which seem to check the conclusions drawn from that set of tests in that the experimental project that was constructed, using an SC-2 oil with a relatively high moisture content in the aggregate in the mixing and put down immediately without drying in the windrow, and which flushed rather badly. The capillary movement carried the bitumen out with it, the free bitumen coming to the top. The top inch or two was very rich and rutted badly. Under exactly the same conditions when using an SC-3 oil it behaved perfectly satisfactorily. There appeared to be no flushing on the surface.

There was some comment about adding dust to offset the light viscosity of some of the oils, and one of the other speakers brought out a point that emphasizes the importance of the viscosity of the oil or fluid binder. In that connection there is involved a question of just what connection there is between viscosity and the oil and stabilization with the small percentage of bitumen. We have suggested that the bitumen is carried by the capillary water, and possibly the higher viscosity bitumen plugs the entrances to the larger pore spaces. This deposition of bitumen at the throats of capillary channels may be the essential factor in stabilization or in at least part of the mechanism, rather than the entire coating of the soil particles, although both of those things, of course, may enter into it.

Mr. Steinbaugh: Have you any further comment on any of the questions?

T. R. Perry: Iowa

Early in the discussion Mr. Gould mentioned a particular section where SC-1 was used and which is now saturated with water. The suggestion has been made that this condition might be cured by adding limestone. Does anyone else have any suggestion to help us out? You understand that this base was compacted originally to Proctor density last fall. It went through the winter, and a test last week indicates a moisture content as high as 17%. The per cent of SC-1 used was about 4%. I have in mind that we could tear this base up and recompact it. Perhaps that is something we will have to do every year or two, but I am just wondering if anyone else can give us an idea of what we might do to permanently remedy this failure.

F. V. Reagel: Missouri

We noticed, in connection with bituminous stabilization, that occasionally we would have stretches that I imagine behave similar to what you mean, but the manifestation of it was in alligatoring and cracking to start with. We found that below we had an extreme moisture condition in the bottom part of the base, but the top dried out crusty and cracked. We discovered that before the completion of the work, there would be occasionally such stretches. We would go back with the blade, break them open, and allow them in warm weather to stand for half a day or so. That would bring the moisture below the optimum. Then we would add moisture to bring it up to the optimum and recompact, and we had no further difficulty.

Mr. Steinbaugh: I am afraid you are going to have to do some experimenting, Mr. Perry, to get the answer.

Has anyone else had any questions or subjects to be discussed in connection with any of the papers or in connection with any of their problems? Have any members of the committee?

G. S. Covert, Jr.: Mississippi

It has been the practice of Mississippi for some years to use hot plant mixed pavements wherever local aggregates were available. We generally used a 3″ base course with an inch and a half Topeka on top of it. I want to know if there is any more cracking where you use a mechanical finisher than in any other type.

Mr. Steinbaugh: Has anyone else had experience with using mechanical finishing, in having accelerated cracking? We have been using mechanical finishing with all of our mixes for five or six years, and started originally about 1929-30, on sheet mixes as well as bituminous concrete or Topeka mixes. There has been some show of tearing during screeding or sheet asphalt mixtures, quite definitely so where the screed was not pre-heated and where too high speeds were used.

I don't know whether any of that has become apparent in stone mixes, but I imagine the tendency would be there to a certain extent if it would occur in the sheet mixes, and in order to eliminate that trouble we have been using a gas device on the finisher to pre-heat the screed, also slowing down the forward action of the machine. In other words, we had to separate the action of the screed from the forward action of the machine, so they would be independent.

G. S. Covert, Jr.: Mississippi

I probably didn't make myself clear. That is interesting. We have been laying 4½″ pavement, you see, and in the course of three to five years it starts drying out and checking badly, in some instances more than in others. I wonder if that checking in thinner pavements, 2″ as against 4½″ or 5″, is more rapid due to drying out.

Mr. Steinbaugh: That has not been our experience. I think that drying out, or loss of ductility and penetration and other qualities in your bituminous mixture, is itself a function more or less of the quality of the asphalt rather than in the thickness of the pavement. I don't think it makes a great deal of difference whether it is one inch or even five inches thick.

Roland Vokac: New Jersey

I will hark back to the first experiences we had in the Michigan State Highway Department back in 1929, when we originally used a mechanical finisher. At the time we adopted a Lakewood screed. This was an ordinary concrete finishing machine. We were using it on sheet asphalt for the first time. The result of that job was that although we got a beautiful looking surface, we were constantly bothered with the appearance of what I might call a herring bone pattern of cracking immediately following the screed. This required considerable back work. That led us to try narrower screeds. At the time we were more or less pioneering in the use of the machine on sheet asphalt paving, and we hadn't developed it to the point where we were applying heat to the screed, but we had figured out that the drag of the screed was sufficient to cause the sheet asphalt mix to tear and, if we were to make it narrower, we would get away from that trouble. In a large measure we did, but as the experience of the highway department has shown since, those mixtures were designed for high Hubbard-Field stability although containing only slightly higher than normal filler content.

Pavements that we made with the finishing machine developed further cracking, but all of it in general following this herring bone pattern. Later pavements of the same mixture that were laid by hand, although they didn't follow this system of cracking, also indicated that they did not contain the proper materials or possibly that they were improperly designed because they also cracked, but where we used the finishing machine cracking followed a definite path of the zigzag motion of the screed.

Mr. Steinbaugh: The question I wanted you specificially to answer, Mr. Vokac, was, in your opinion, do you believe there is any difference in the loss of the qualities of the bituminous material in a thin or a thick pavement? You have done considerable work along that line.

Roland Vokac: New Jersey

No, I do not believe so. In an investigation completed last year and reported in part before the Association of Asphalt Paving Technologists last December in Memphis attention was called to the fact that in cracking pavements I observed the only difference that really existed

in the physical make-up of the mixture as compared to good pavements was a difference in density. The average composition of the good mixtures and the poor mixtures was the same as to mixture proportions, but the mixtures that had cracked invariably showed less density than could be obtained in the laboratory, whereas the good mixtures had density equal to that obtained in the laboratory.

Laboratory work has shown us also that the hardening of the asphalt is not affected by the size of the specimen so long as the densities of the specimens that you are comparing are the same; further, when a rich mixture is compared with a lean mixture, the hardening of the asphalt progresses at very much the same rate providing that your mixtures are all equally dense or impermeable. So, on these bases, I don't believe you will find any difference in the cracking tendency as between the thick pavements and the thin pavements. If the asphalt is hardening because the pavement lacks density, it would harden just as rapidly if you had a thin section that was of equal density.

Mr. Steinbaugh: Except that there is one thing that has not been brought out. Perhaps it is a little easier to secure maximum density in the thinner pavements than you would in the thicker ones, but assuming they were equal, I would say from our experience it would make no difference. Mr. Vokac, that is a very good point. Is there anyone else on the committee that has a subject or a point he would like to bring out?

J. R. Phillips: Wyoming If I may, I would like to go back to one thing that was mentioned this morning. I am sort of interested in a statement made by Mr. Fahnestock of North Carolina, and that was with regard to the sand asphalt retread mixes to level up an old pavement, in which he stated they were stable even when twelve inches thick. I would sort of like to know what type of oils he used, how he mixed them, and if built in layers, how many layers, and just the lowdown on it.

T. V. Fahnestock: North Carolina The retreads are made of sand and, when used for superelevating curves, are usually mixed with RC-1 or RC-2, and they are generally mixed by just placing the sand on the road, mixed with a motor grader or retread paver. It is essential that the moisture and the solvent material in the mix be practically evaporated before the mixture is spread if you are to obtain stability. There is no trouble in determining when you are going to get that stability. If you spread it too quickly you will soon find out you haven't got it and you will have to tear it up for additional aeration.

As for the design of the mix, the amount of bituminous material that is used in the mix of course depends on the grading of the sand. As I stated this morning, there is no formula which we found entirely satisfactory. We make up trial mixes and observe them, then we make the mix on the road and we tend toward the lean side; if the mix should prove to be slightly lean the only trouble that will result will be a sanding off of the surface, which can be easily corrected by a tack coat on top, or by the addition of more bituminous material.

. R. Phillips: Wyoming — There is just one point that is not clear to me, Mr. Fahnestock, and that is, if you were making a 12″ lift would you make that all at one time or in several layers?

C. V. Fahnestock: North Carolina — It isn't practical to mix enough material with a blade to make a 12″ lift at one time, so it is more satisfactory to bring that up in about two lifts. One of those lifts is shaped and rolled either with rubber-tired equipment or with a flat wheel roller. We don't use the sheepfoot roller on the sand-asphalt mix. The other layer is put on top of that and finished in a similar manner.

When we are planning to place a retread over an old road we generally go over that road and build up the curves first, and then we come back with a retread over the entire road, also over the curves, and it gives a uniform surface over the entire roadway.

Mr. Steinbaugh: Who else, now, has a subject that he would like to start?

J. W. Powers: Arizona — In one of these papers that you have here you make the comment that oil mixes are quickly analyzed, although for road mixes the moisture determination on the aggregate would be required. I am interested in knowing what kind of equipment you use for making that determination.

J. G. Schaub: Michigan — A stove pan and scales to get the moisture content by the ordinary method of evaporation.

J. W. Powers: Arizona — That can't be done on oil.

J. G. Schaub: Michigan — I thought you meant before the oil is applied, to determine the moisture of the aggregate.

J. W. Powers: Arizona — I thought from this that it was done on both; aggregate can be done on a stove but it isn't so easy to do it on the oil mix.

Mr. Steinbaugh: Our man from the laboratory was on his way down here and had a death in the family and had to go back. I don't think I can answer that for you.

Prof. W. S. Housel: Michigan

I can attempt to answer the question. That particula comment questioned by Mr. Powers referred to the o aggregate construction where the aggregate is dried befo the mix. Moisture determinations are not ordinarily mad in the field on plant mix after the mixture has bee laid or prepared for laying. Determinations of moistu are made on the aggregate before it is mixed with th oil by drying to constant weight on a gasoline stov and computing the moisture content from the differenc in weight before and after drying in the usual way. Th moisture content of field mixtures referred to in M Brown's paper were determined on samples taken t the laboratory and subjected to the usual distillatio test.

J. W. Powers: Arizona

Do you have any facilities in the field in case, after yo have dried it out, you should have a rain and have t dry it out again?

Prof. W. S. Housel: Michigan

I don't believe so. If it rains on the aggregate it wi have to be dried out and tested again. If it rains o the mix itself I am not familiar with the method the would use in the field in that case.

J. W. Powers: Arizona

That is what I was interested in.

Mr. Steinbaugh:

All of our mixture is plant mix. We don't do any mixed in-place work at all, so the mixture just wouldn't b laid under such conditions.

E. L. Wales: Arkansas

We have field apparatus in our state that we use, and it has been quite satisfactory. I wonder if Mr. Powers i speaking of cut-backs or slow-curing oils.

J. W. Powers: Arizona

Either one.

E. L. Wales: Arkansas

For our cut-back mixtures we use a very simple distilla tion apparatus, just two glass flasks and a condenser We put our sample from the windrow in one flask—the other is a steam flask. We run steam through the sample and collect the volatiles through the condenser. After all the volatiles have been driven off, the steam is dis connected and the sample is heated until all moisture has been driven off. With the total loss in weight of the sample and the amount of volatiles, we compute the percentage of moisture in the mixture. This apparatus has been checked with synthetic mixtures in the laboratory and results have been quite satisfactory.

Mr. Steinbaugh:

Is there anything else?

J. H. Barnett: Tennessee

A suitable method that might be tried is the method for determining the volume of moisture in a sample. By weighing out the sample and adding naphtha as in the

usual Dean and Stark method, as prescribed by the A.S.T.M., determination of the moisture content of soil mixtures with asphalts, tars or any oils that are volatile in heat can be effected.

Mr. Steinbaugh: Are there any other subjects that you would like to have brought up relating to the construction of bituminous pavements?

J. G. Schaub: Michigan

Mr. Steinbaugh, I would like to know what experience some of the states have had in surface treating highly stabilized gravel surfaces. As Mr. Steinbaugh remarked, a year or so ago we started building pre-stabilized gravel bases with the use of chloride, and we want to surface treat them to get rid of the dust nuisance. We tried out several methods of surface treating. Our method in the past on ordinary gravel roads before we started stabilizing was to use the ordinary prime coat material, such as an MC-1, or TC-2, and then applying a heavier material to hold the chips. In using that method on the stabilized gravel surfaces, we don't get any penetration of the prime material, and I am rather interested in knowing what experience some of the other states have had, whether they use a prime and what type of prime in surface treating stabilized gravel.

Mr. Steinbaugh: Has anyone that has been doing gravel stabilizing had any experience in regard to that, getting penetration of bituminous materials into the stabilized surface?

J. W. Powers: Arizona

I presume that you mean by surface treatment the application on top of a gravel base of a certain type of oil to the extent of somewhere between half a gallon to three-quarters gallon to the square yard, and allowing it to penetrate.

We have done probably 150 or 200 miles of that in our state and we do not have any trouble with the application of the oil providing the material is slightly damp.

Mr. Steinbaugh: What kind of oil do you use?

J. W. Powers: Arizona

We use SC-2 and MC-2. If you get it too wet it has a tendency to hold the oil up.

Mr. Steinbaugh: You dampen the surface of the stabilized gravel?

J. W. Powers: Arizona

Yes, just prior to the application of the oil, and in that way we get anywhere from one-half to three-fourths of an inch penetration, which is as much as you should expect, because you don't want to distribute the oil too far into the base. The moisture in our particular case is a benefit rather than a hindrance.

J. G. Schaub: Michigan

Is that a gravel surface stabilized with clay and chloride?

J. W. Powers: Arizona

It is the natural gravel we have in our country, crushed to an inch maximum size, carrying probably not to exceed 10% minus 200. The P. I. of the material that we use would probably not exceed 4 or 5, and in most cases would be around zero up to 2.

Mr. Steinbaugh: Thank you.

J. H. Swanberg: Minnesota

We have found in our stabilized gravel bases where we have primed those surfaces within about three weeks (the percentage of clay will vary, probably depending on the grade of clay, carrying probably 10 to 15% of clay with a P. I. of probably 10 or 12, and priming with either MC-1 or TC-1) that we get a penetration of about a quarter of an inch.

I think this is true, that if you have excessive moisture in the base you are not going to get the penetration you would like to have or should have. In some laboratory work we found, as Mr. Powers has stated, that by adding moisture just prior to the prime coat we obtained increased penetration. Where we normally get a quarter of an inch, by moistening we might get 5/16" or 3/8".

Along that line, we had a clay stabilized job last year and we came in immediately afterward and primed it with MC-1, and put a tar seal coat on top of that, sand covered. Some of those sections on that job pitted rather badly, and because of lack of maintenance that condition was aggravated. The engineer had made a very careful study of the length of time the MC-1 was down before the tar seal coat had been placed on it. In some sections the prime had been down for a period of nine days prior to the placing of the seal coat material, and it was indicated on the basis of his study that the longer the prime had been down, the more durable was the seal coat.

In those sections where we had a nine-day interval between the prime and the seal there was positively no pitting over that stretch, and where the seal had been put on the next day following the prime there was considerable pitting.

J. W. Powers: Arizona

In that connection I might add that the usual type of surface treatment we put on is very similar to the one Mr. Swanberg has just described. We use a half gallon as an initial application and use the extra quart as a seal coat, and we do not apply the seal coat until there has been an absorption by the base of all of the oily material that has been placed on it prior to that time. Sometimes it takes two days, sometimes three or four.

Mr. Steinbaugh: But the difficulty Mr. Schaub is having is getting penetration from the original application. Has anyone been able to get satisfactory results with any kind of emulsion?

C. R. Perry: Iowa

Mr. Chairman, about three weeks ago we used an emulsion, a rather heavy emulsion, and in order to secure any penetration whatever it was necessary to wet the subgrade; that is, it was necessary to wet the stabilized base. But even then we really did not secure any penetration to speak of. The asphalt simply laid on top, and in order to proceed with the surface treatment it was necessary to place cover aggregate over this prime to blot it up.

As for the penetration on some of these dense stabilized bases, the depth of penetration will be less than on the more open gravel surfaces. We use .4 to .5 gallon of prime per square yard on the open mixes, and on the denser ones from .2 to .3 gallon per square yard.

Mr. Steinbaugh: Thank you.

Prof. W. S. Housel: Michigan

I would like to ask Mr. Powers a question. Judging from the comments of Mr. Swanberg and Mr. Powers, one of the essential elements involved in the penetration of the seal coat would be that the aggregate base itself be dry before the moisture is sprayed on the surface. If that is the case, would you subscribe to the idea that the essential mechanism there is that the moisture that is applied prior to the seal coat was being drawn by capillary action into the base and carrying the bitumen with it?

J. W. Powers: Arizona

I will answer that question this way: It is my observation, Mr. Housel, that it is due to the fact that you cut the dust so that it doesn't have any tendency to ball up the oil, and allows it to go down through. As I see it, it filters down through there, whereas if you allow it to be dusty it balls up, your fines block the oil and will not allow it to go down.

Prof. W. S. Housel: Michigan

My suggestion would check up with what Mr. Brown found in his paper about the necessity of the drying period, where you had excess moisture in the mix, as a means of distributing the bitumen. Movement of capillary moisture appears to be an essential part of the mechanism of depositing the bitumen and sealing the mixture. Your observation would seem to indicate that same idea, as would Mr. Swanberg's.

W. W. Zass: Arkansas

The matter of a limiting value for plasticity index of soil-bound base courses has been lightly touched on and various ranges have been given. One state advised that they were using a maximum value of 6, another state of 12, and another state of 15. It would appear to me that there could well be a definite expression among the various personnel present as to what they would consider a conservative maximum value of that factor.

Also, another question while I have the floor, in regard to the solvent content that the states are allowing to be incorporated in the mixture at the time of spreading. As I understand the general practice, from conversation and from discussion, many of the states are spreading their dense graded mixtures immediately after the application of the liquid asphalt, that is after it has been properly mixed with the mineral aggregate, with no governing value fixed as to the amount of solvent that may remain in the mixture. I would like to hear both these points more fully discussed.

Mr. Steinbaugh: Who will volunteer on either of those questions?

C. T. Davis: Texas — As for the P. I. on base materials, we have found in Texas by actual test on bases that have been down many years, if we get a plasticity index in our caliche and crushed stone bases of more than 15 we are very liable to run into trouble. As a consequence, while we have some bases that have a higher P. I. than 15 that have stood up for several years, we have a specification limiting now of a P. I. of 15 on caliche and crushed stone base material. On the gravels we lowered that to 12. We don't have any lower factor for the P. I., but we think we have found out that the lower factor of the P. I. is no determination of whether the material will set up or not. I can recall one job on which we had two pits, one of which had a P. I. of zero and the other of 14, and both were set up in the same way, and I believe the one with the zero factor is standing up better than the other.

Most of our work is being set up with RC's on plant mixes. You have to lay it down to make a pavement on it, and the amount you leave in there depends on the temperature and humidity at the time you lay it down. If it is cold you have to lay it down with more volatiles in them than when they are hot, because the viscosity of the mix is such that it is not workable if you keep on blading it. That is a bad point, so we try to do that work in hot weather if possible.

W. W. Zass: Arkansas — Does your specification cite a maximum volume of solvent that may be carried?

C. T. Davis: Texas — The specification for the cutback itself does, yes. There is no specification with regard to the mixture.

Mr. Steinbaugh: Thank you, Mr. Davis.

C. T. Davis: Texas — I think it would be best if you could get it all out before you lay it down, but if you did you could not lay your pavement down.

J. H. Swanberg: Minnesota — I think there is a point that is worthy of consideration relative to this item of P. I. When we started making soil tests in Minnesota we followed the procedure out

lined in Public Roads, Vol. 12, No. 5, dated July, 1931. The lower liquid limit and plastic limit values were determined by the hand method. Since then a machine has been developed and adopted for making lower liquid limit determinations. With the calibrated liquid limit machines we got values much lower than by the hand method, thus lowering the P. I.'s and causing confusion.

We have had considerable discussion on this problem with the Bureau of Public Roads because if we adopted the machine method it would mean lower P. I.'s and consequently a change in our specification limits would be necessary.

C. T. Davis: Texas — That machine you are talking about—has anybody got one that will really work?

J. H. Swanberg: Minnesota — We get values from it, but we prefer to use the hand method.

C. T. Davis: Texas — That is what everybody else does that I have run into. Has anybody one of those that really works? If so, we would certainly like to know it, if it is really good.

V. G. Gould: Iowa — Regarding the P. I., what I have to say now is a personal comment, and probably wouldn't stand up long under argument, but it is my thought that we are giving far too much importance to the P. I.'s. What we are interested in primarily is the cementing qualities of the material which we are stabilizing, and the P. I. becomes important only in its maximum value in avoiding the highly plastic conditions in the presence of undue moisture.

G. S. Covert, Jr.: Mississippi — We find in Mississippi that you can use a much higher P. I. with a well-graded material than where the material isn't so well graded. I have in mind a surface treatment project, a triple surface treatment where we used an emulsion, where we had a P. I. of 18, and in five years we have not had any maintenance.

Leon Gottlieb: Alabama — In regard to the plastic index, I can state that in Alabama in our base course material the higher P. I.'s are successful only when subgrade conditions are favorable, and when heavy plastic sub-soils are encountered we find it necessary to lower the P. I. in the base course material in order to prevent failure.

In checking projects already finished, we found on many projects where the base course materials have a P. I. of around 12, and occasionally 15, that our subsoils conditions have been most favorable. On the other hand, with high P. I.'s and bad sub-soil conditions we have had lots of trouble. Apparently, there should be a correlation between the gradation, the soil fines and the P. I.'s. Where you have a minimum amount of soil fines

you can have a higher P. I. and still not have any trouble with your base courses. Where you have a maximum amount of soil fines, that is, your 40-mesh material and have a high P. I., you may run into difficulty.

T. R. Perry: Iowa — In regard to Mr. Zass' question of what the limits of P. I. should be, the Bureau requirements for the material we have in Iowa are zero to 6. I would dislike turning down a contractor's job because the P. I. on some of that material was 7. If we could have that raised, to say 8 or possibly 9, we would come under the wire in pretty nice shape.

W. W. Zass: Arkansas — Would you still consider that satisfactory, Mr. Perry?

T. R. Perry: Iowa — I believe we would.

O. L. Stokstad: Michigan — I think there has been too much emphasis placed on the subject of P. I. The significance of the P. I. apparently differs with various zonal soil groups. It seems reasonable to expect that a similar P. I. value would not indicate similar properties when comparing the Podsols of Minnesota with the Chestnut soils of the West. I would like an expression from Dr. Winterkorn on the subject.

Mr. Steinbaugh: Is Dr. Winterkorn still here? I believe he is not at the meeting. Mr. McLeod, would you have anything to say in regard to that subject of P. I.?

N. W. McLeod: Saskatchewan — Only to say that the P. I. of clays of the same particle size distribution in different parts of the United States and Canada could vary over quite wide limits, depending on the soil-forming agencies in each area. Therefore, the P. I. of podsol, prairie, chernozem, and the red clays of the southeast could be quite different even though of the same particle size distribution. Consequently, the P. I. of mixtures of a given gravel of good gradation stabilized with a given percentage of these clays could vary over a range of several percentage points.

Mr. Steinbaugh: Has anyone else?

Prof. W. S. Housel: Michigan — I have to get up again but I have been waiting for Dr. Winterkorn. Apparently he has gone. I think he would have had something to say, at least in response to Mr. Stokstad.

First I want to say a word about Mr. Davis' question: Is anyone using one of these machines and getting satisfactory results with it? Are you referring to the liquid limit machine?

C. T. Davis: Texas — Yes.

Prof. W. S. Housel: I thought you might be referring to the Plastograph. We have been using the liquid limit machine for several years. However, we use ten blows instead of twenty-five blows, so I can't say we are using it in a standard way by any means. We used ten blows by hand when using the hand method, and in calibrating standard samples from the Bureau with the liquid limit machine, of which we have several, we check them pretty well on ten blows. Consequently, we have been using that in spite of the fact that the standard method calls for twenty-five blows. I believe the test is open to some question.

I think it would be interesting to this group to know that this was a very live topic of discussion at the last A.S.T.M. meeting, where they were attempting to change the standards of the test and the concensus of opinion was to leave the test alone, recognizing that it wasn't ready for final standardization now. It was decided to wait until we had more information on what P. I. meant and what the proper standard should be, so that that test is very much to be questioned and changes will be made, I think, in the next few years.

I agree with Mr. Stokstad, that we are placing too much emphasis on the P. I.'s. I don't know how many of the gentlemen here have heard the discussion in previous meetings, but I would judge from the consensus of opinion among those here that the P. I. test is not a satisfactorily designed test for the stabilized mixture itself. It may be all right as a quality test for the binder, but the best criterion for the mixture itself is to go back to the old fundamentals of not over-filling the voids in the aggregate with clay. That is what happens when we try to produce a certain P. I. for a stabilized mixture with a low P. I. binder. We fill the voids up so full of silt and poor clay that it is extremely detrimental to the mixture, and I believe Mr. Powers and several other men who have talked on that subject have checked on that. On the other hand, you may have a zero P. I. for the mixture, but if the voids aren't over-filled, the mixture may be quite satisfactory.

Mr. Steinbaugh: Is there anything else you would like to have discussed today? If you haven't anything else that you would like to have brought up, I would be inclined to dismiss the meeting and say we are adjourned.

Papers Prepared

on

Construction of Bituminous Surfaces

for

The Montana National Bituminous Conference

Gravel Stabilization With Michigan Slow-Curing Oils

J. D. BROWN,
Engineer of Bituminous Research
Michigan State Highway Department

An abundance of readily accessible and relatively cheap Michigan slow-curing oils has prompted investigations by the Michigan State Highway Department towards the satisfactory usage of these oils.

Since slow-curing oils have been used successfully by several states in soil stabilization, their usage in a similar capacity in Michigan suggested itself. However, since most of our trunk-lines are paved or graveled, soil stabilization, as such, by road mixing did not warrant an investigation.

In the last year or so a very appreciable tonnage of clay-chloride stabilized gravel has been produced both for secondary road relocation or new construction, and for increasing the thickness of existing gravel roads. As fast as funds permit, these gravel roads are utilized as bases for bituminous surface treatments, bituminous concrete, or oil-aggregate surfaces. As an alternate type of base construction it appears that gravel stabilization with slow curing oils in an amount sufficient to reduce capillary water absorption to a minimum should be an improvement over the highly capillary clay stabilized bases and should aid materially in prolonging the life of the bituminous wearing surface.

To be economically feasible two conditions should be fulfilled; first, the materials cost of oil stabilized gravel should not greatly exceed the cost of clay-chloride gravel stabilization; and second, little or no additional equipment beyond that used in the present stabilizing plants should be required. An answer to the first requirement could be made only after undertaking a laboratory investigation to determine the approximate amount of oil needed. As to the second requirement—pug-mill mixers, and clay and gravel feeders are now required, so the only additional equipment needed would be a bitumen pump, and in a number of plants these are now available. Whether a drier would be necessary also depended upon the laboratory findings, since no information was available which would predict the value of oil in reducing capillarity when mixed with aggregate containing from 0 to 6% water. A further question to be answered was the viscosity range of bitumen which could be readily mixed with damp aggregate.

The first part of this report will be devoted to the laboratory in vestigations, and the second part to the application of the laborator findings in the construction of an experimental test section of road.

CAPILLARY WATER ABSORPTION BY OIL-GRAVEL MIXTURES

Test Procedure

The usual procedure in preparing to perform capillarity tests is t separate the fines from the aggregate and compress the damp mixture in molds from which samples may be forced intact. It was obvious tha a modification of the procedure was necessary for the granular grave mixtures, low in fines and bonding material, since there is no assuranc that tests made on a small portion of a mixture are at all comparabl to those made on the entire mixture. The procedure finally adopted is t compress a given weight of the mixture in 6-inch cylindrical molds, inches high, under which are placed several thicknesses of white blottin paper resting on flat iron plates. The mold assembly is then placed in constant level water bath so that the lower blotting papers are in contac with the water. At the expiration of the absorption period, usually days, the material is forced from the mold directly onto a balance pa and weighed immediately. The absorption is expressed as a percentag of the weight of the briquette before absorption.

Other variations in test procedure which have proven desirable ar the methods and temperatures used in drying the briquettes before sub mitting them to the absorption tests. These points will be discussed i conjunction with test results.

Materials Used in the Tests

The gravel used in the following capillarity tests was taken from a large stockpile of material which was to be used with admixtures o clay and chloride to produce stabilized gravel. The gravel was, according ly, high in the portion retained on the No. 10 sieve, so that for the pur poses of this investigation it was separated at this sieve into coarse and fine components. These were recombined for each individual briquette (to avoid segregation as much as possible) to give 50% material re tained on, and 50% passing, the No. 10 sieve. The grading of the gravel was as follows:

Passing ¾ in. sq. screen, retained on ⅜ in. sq. screen	18.3%
Passing ⅜ in. sq. screen, retained on No. 10 sieve	31.7%
Passing No. 10 sieve, retained on No. 40 sieve	30.2%
Passing No. 40 sieve, retained on No. 200 sieve	12.9%
Passing No. 200 sieve	6.9%

The analyses of the oils and tar used in the investigation are given in the appendix. The specification for the soil stabilization emulsion is also given, the complete analysis not being available.

Effect of Test Conditions on Results

As previously noted, it seemed advisable to investigate the effect of temperatures and methods used in drying the absorption specimens. Normal-

the procedure is to oven dry the samples at 60° C. to constant weight. his temperature was no doubt chosen as the highest ordinarily reached a pavement. For the greater part of the construction season, however, e road temperature is much lower. Furthermore, normal evaporation excess water from a road must, of necessity, be from the surface only, hile moisture from laboratory briquettes evaporate from the sides and ttom as well. It is of utmost importance that the significance of results this type of laboratory test, made under conditions differing from ose encountered in field work, are not misinterpreted. Such an example ill be given in the discussion of the field tests.

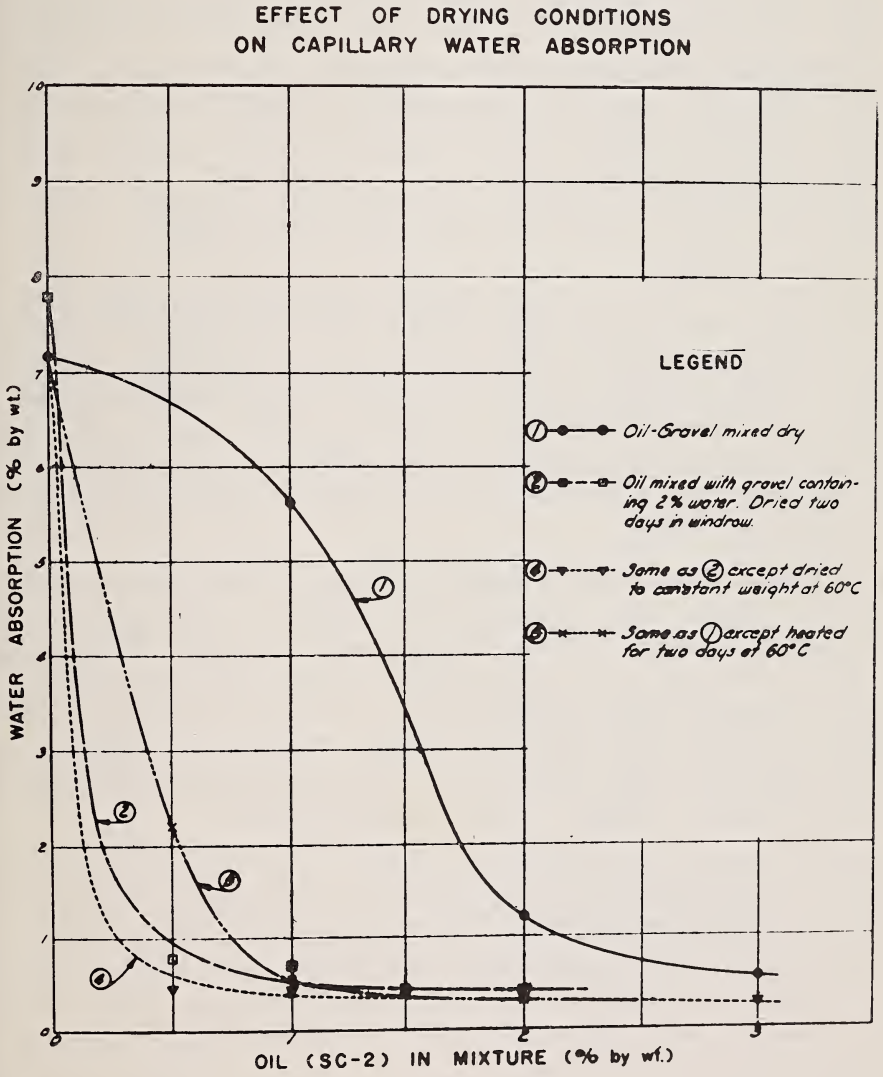
EFFECT OF DRYING CONDITIONS
ON CAPILLARY WATER ABSORPTION
WATER ABSORPTION (% by wt.)
OIL (SC-2) IN MIXTURE (% by wt.)
LEGEND
① Oil-Gravel mixed dry
② Oil mixed with gravel containing 2% water. Dried two days in windrow.
④ Same as ② except dried to constant weight at 60°C
⑤ Same as ① except heated for two days at 60°C

FIGURE 1

Curve 1, Figure 1, shows the capillary water absorption (for the usual 7 day period) of dry gravel mixed with various percentages of SC-2 oil. It will be demonstrated later that the shape of the irregular curve is due to the non-uniform coating of low percentage of oil on dry aggregate.

As a check on the accuracy of the water absorption tests, voida determinations were made on gravel with zero oil content. The tot voids averaged 16.8% by volume which is equivalent to water content 7.1% by weight—very close to the absorbed water value. It is evident th the pores in the gravel are completely filled with water.

To prove that the size and shape of the mold selected had no effe upon results this test was checked with a cylinder 4 inches in diamet and 7 inches high. The absorption obtained was 8%. The effect of segreg tion of such widely varying particle sizes during compaction was eval ated, individual determinations on gravel mixtures with zero oil conte showing a maximum variation of 1% absorption. Therefore the value 8% may be considered a reasonable check on previous determination If the absorption time periods were appreciably shorter than 7 days, th height of the molds would no doubt influence results. It was determine that the maximum absorption of bitumen-free gravel of this gradatio is attained in approximately one day, so that a 7 day test for all pract cal purposes appeared sufficient for maximum absorption with sma amounts of oil which increase the time interval noticeably.

Following the above outlined capillary absorption procedure, a secon set of samples was tested after having been prepared by mixing th aggregate with 2% water, to facilitate mixing with oil and to aid in con paction, and then drying in an oven at 60° C. to constant weight. Th absorption results, shown in curve 4, Figure 1, indicated such a marke improvement over those of curve 1 that the question arose as to th relative or combined influence of the moisture content and temperatur variables on test values.

Eliminating the moisture variable entirely, a third set of specimen prepared by mixing the dry aggregate with oil as for curve 1, wer heated for 2 days at 60° C. The results, shown in curve 5 are very nearl as good as those shown in Curve 4. It is evident, since the data show in curves 1 and 5 were obtained under identical conditions with th exception of heating, that there is a capillary flow of the relatively flui SC-2 oil within the mass during the two day heating period, resultin in a more effective coating of the mineral particles.

For the purpose of eliminating heating temperatures above normal a fourth set of samples, each containing 2% water as in curve 4, wer prepared and allowed to air-dry for 2 days in a window on an iro table top, turning over several times daily as in road blading. Moisture tests on duplicate samples at the end of the air-drying period averaged 1½%. In spite of the moisture present the absorption values (Curve 2) ar very close to curve 4. The failure of the attempt to remove the greate part of the moisture present leads to the conclusion that the manipulatio of the mixture has aided in coating the film of water surrounding the mineral particles with oil films. Further mention will be made of these results in the next section.

Effect of Moisture in Mixes on Water Absorption

When it was finally decided to construct an experimental section of oil-stabilized gravel bases and lay the material with a finishing ma-

ine, it was necessary to further investigate the effect of moisture in the gregate, in percentage up to 5% which is common in stockpiled gravel, both mixing with oil and water absorption.

The results of the further investigation of the effect of moisture are ven in Figure 2, where some of the curves already shown are repro- ced for ready comparison. The absorption data given in curve 3 were tained from samples prepared as in curve 2, with 2% water, sub- cted to absorption immediately without an intervening drying period.

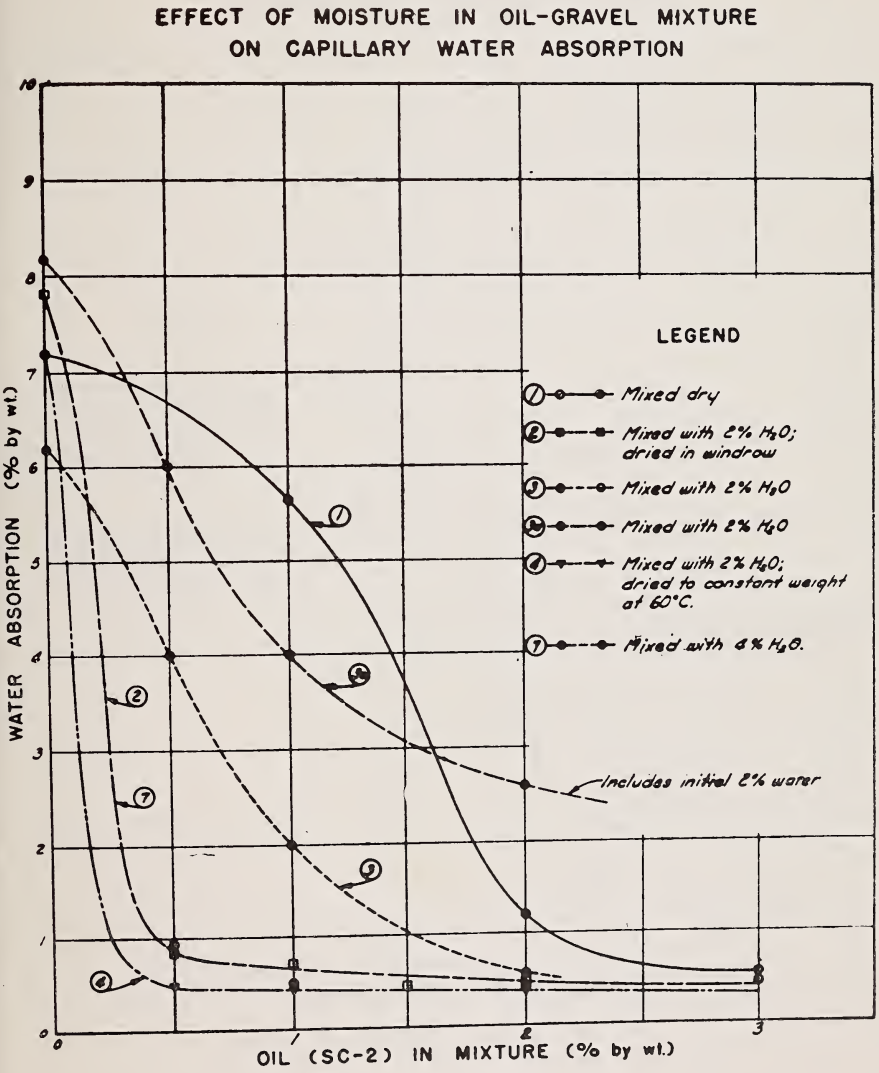
EFFECT OF MOISTURE IN OIL-GRAVEL MIXTURE
ON CAPILLARY WATER ABSORPTION
WATER ABSORPTION (% by wt.)
OIL (SC-2) IN MIXTURE (% by wt.)
LEGEND
1 Mixed dry
2 Mixed with 2% H2O; dried in windrow
3 Mixed with 2% H2O
3a Mixed with 2% H2O
4 Mixed with 2% H2O; dried to constant weight at 60°C.
7 Mixed with 4% H2O.
Includes initial 2% water

FIGURE 2

The total amount of water present after the absorption period is, of course, 2% more than the absorbed water and is as shown in curve 3a. The somewhat higher zero oil content absorption also checks with the volume of voids which averaged 20.4%, which is equivalent to 3.8% water by weight. This indicates that compaction is a little more difficult with 2% moisture present than when dry, although there was noticeably less tendency toward segregation.

The absorption percentages shown on curve 3, especially for o contents below 1½%, are materially higher than for those specimer which were air or oven dried, after mixing and compacting, and ar indicative of results which may be expected when the mixtures are lai wet on the road. Above approximately 1½% oil content the water absorp tion, over and above the initial amount present, compares favorably wit dried mixtures.

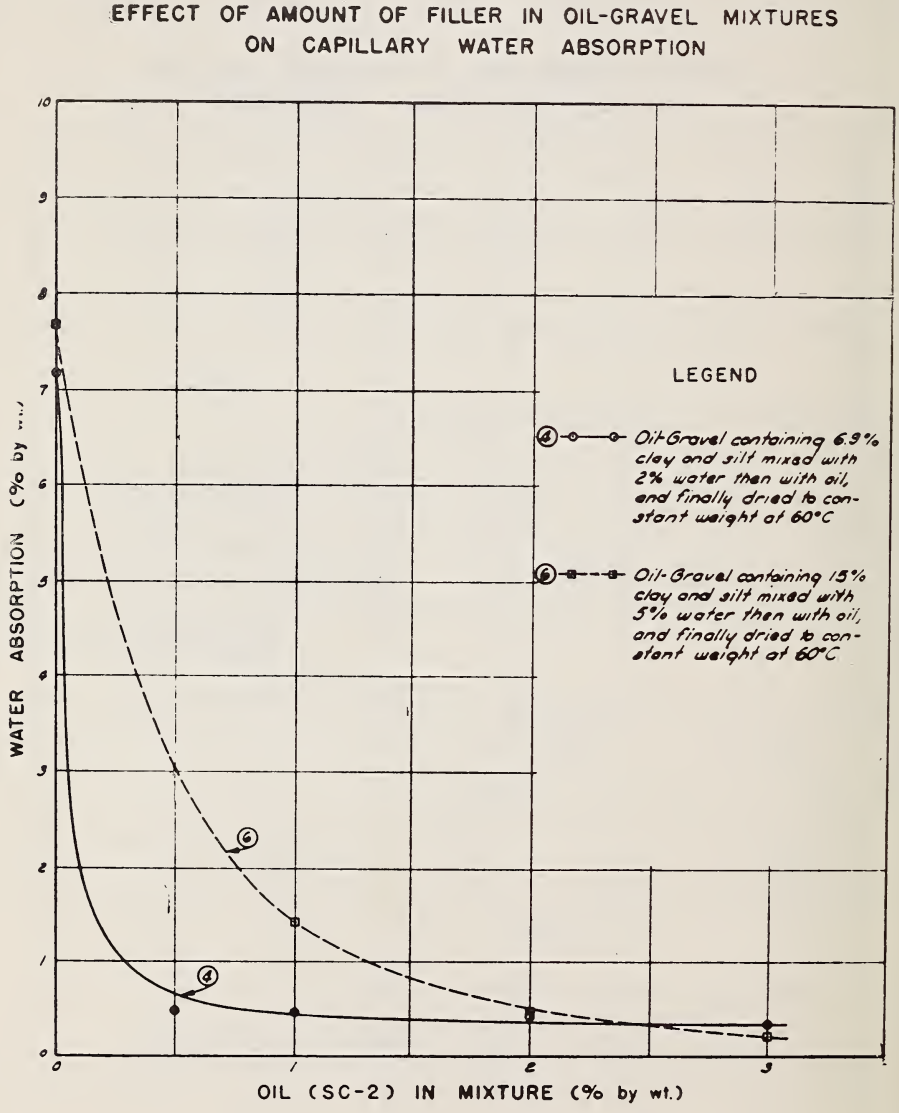
EFFECT OF AMOUNT OF FILLER IN OIL-GRAVEL MIXTURES ON CAPILLARY WATER ABSORPTION
WATER ABSORPTION (% by
OIL (SC-2) IN MIXTURE (% by wt.)
LEGEND
④ Oil-Gravel containing 6.9% clay and silt mixed with 2% water then with oil, and finally dried to constant weight at 60°C
⑥ Oil-Gravel containing 15% clay and silt mixed with 5% water then with oil, and finally dried to constant weight at 60°C

FIGURE 3

So little evaporation was obtained in the air-drying of the mixture containing 2% water that another set of samples were prepared containing 4% water. These were exposed to air-drying conditions as in curve 2. The absorption data, and the moisture content after drying, was practically the same as with 2% water. Absorption values are shown on a common curve 2 and 7. It thus appears that any moisture content above 2% may be removed by blading on the road down to a minimum of about 1½%.

The choice of the proper percentage of bitumen to counteract detrimental capillarity in such mixtures therefore depends upon the moisture content of the aggregate and the method of handling on the road. Under ideal conditions from ½ to 1% of oil appears to be sufficient for this purpose. If, however, the aggregate is dry, the efficiency with which the low percentages of bitumen is distributed is not sufficient to produce the desired decrease in capillarity.

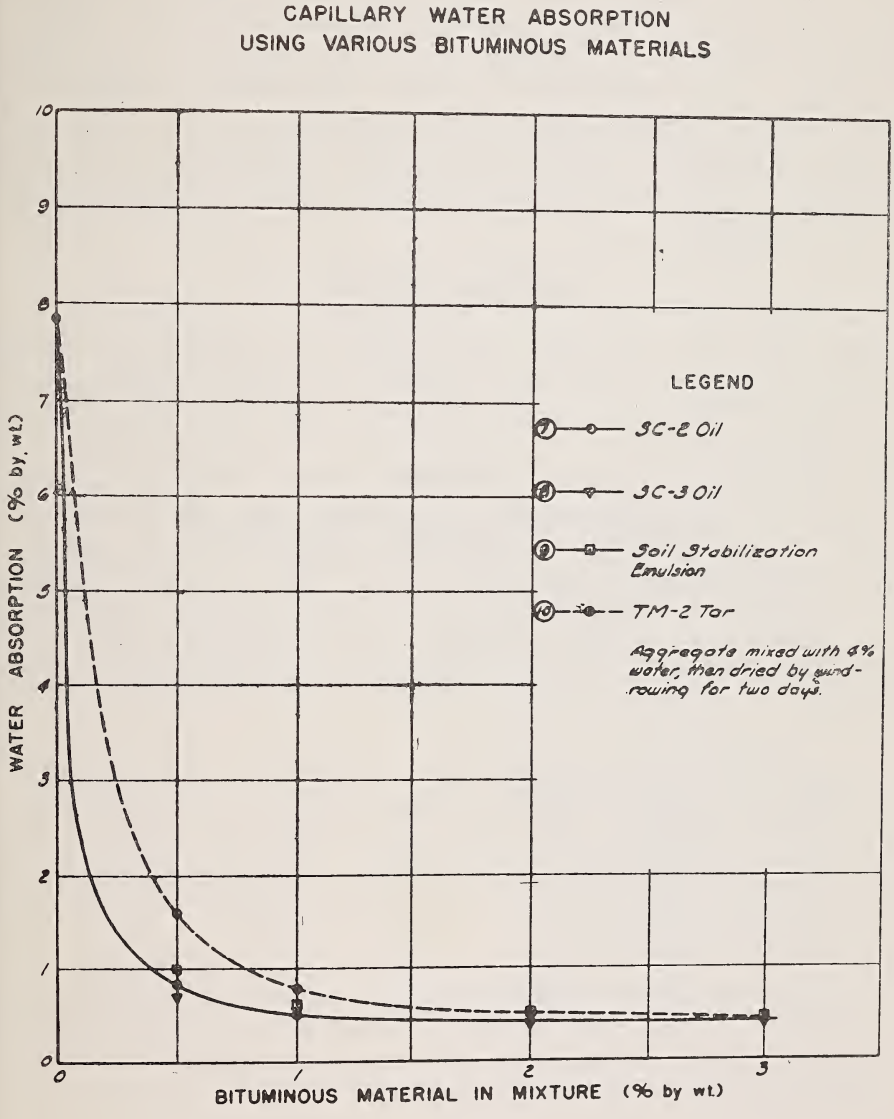
CAPILLARY WATER ABSORPTION
USING VARIOUS BITUMINOUS MATERIALS
WATER ABSORPTION (% by wt.)
BITUMINOUS MATERIAL IN MIXTURE (% by wt.)
LEGEND
SC-2 Oil
SC-3 Oil
Soil Stabilization Emulsion
TM-2 Tar
Aggregate mixed with 4% water, then dried by windrowing for two days.

FIGURE 4

The most significant result of the investigation appears to be that illustrated by curves 3 and 3a in Figure 2. when no opportunity was provided for drying previous to measuring the absorption, the characteristics of the mixture were very little superior to those of the original aggregate. The fact that evaporation involving capillary movement of the moisture appeared to be the essential factor in the decreased capillarity obtained in most of the tests suggests the mechanism involved.

It is recognized that as moisture evaporates from a porous mass the water gradually retreats into the smaller void spaces near the points of contact of particles which are the throats or entrances to the larger voids. The remaining moisture may be more firmly held and is only removed at higher temperatures. In any event the bitumen carried by the water may be likewise drawn into the throats of the various capillary channels and there attach itself to the particles as excess moisture is removed. Bitumen deposited in this manner is in the most strategic location to prevent subsequent capillary flow and incidentally supply the maximum stability. It is suggested that this mechanism may supply an explanation of the effectiveness of small percentages of cohesive agents which are being currently used in soil stabilization as it is not entirely clear how distribution in films of uniform thickness may do so. Some observations in the field discussed later, also favor this special distribution of the bitumen.

Aggregate Grading

Very little has been done on the effect of varying the gradation of these mixtures. It is considered advisable to avoid the high amounts of clay, usually from 12 to 16%, ordinarily used in stabilized gravel. Clay is highly hydrophilic, and possesses natural bonding properties which would presumably be destroyed by the oil but due either to its enormous surface area or the large number of particles the effectiveness of the oil might not be sufficient to restore the equivalent of its original cohesion. A comparison between high and low filler content mixtures are shown in Figure 3 where curve 6 shows the absorption obtained when the clay is increased to 15%. It will be noted that 2½% oil is necessary as against ½% oil in the low filler mix for equal efficiency in combating absorption.

It is, therefore, recommended that oil-stabilized gravel mixtures be well graded, and contain 50-55% of sizes passing the No. 10 sieve, and from 4 to 8% passing the No. 200 sieve. If additional filler is required a fine or loamy sand should be used. Such a mixture will possess ample internal stability and with 1½ to 2% of oil present may be handled as an oil aggregate mixture.

It is realized that the suggested oil contents are insufficient to provide the resistance to wear and abrasion required of a wearing surface, and it will be necessary to promptly seal coat these bases or cover with a thin wearing surface such as oil-aggregate.

Type of Bitumen

Based on the assumption that almost any petroleum or tar product possessing water-repellent properties would serve the purpose, all the data presented thus far have been obtained from mixtures containing SC-2 oil. To check the accuracy of this assumption, additional samples were prepared using SC-3 oil, TM-2 tar, and soil stabilization emulsion mixed with aggregate containing 4% moisture. The mixtures were allowed to air-dry as before for 2 days. Moisture tests on these mixtures after this period of drying varied between 1 and 2%, averaging 1.6%.

Absorption results, shown in Figure 4, for the petroleum products are practically identical. Absorption values for the tar mixtures are but slightly higher, so, except for some practical considerations which will be discussed under field tests, there appears to be little choice in the type of bitumen used to reduce capillarity.

FIELD TESTS

The results of the laboratory experimental work appeared so promising that a 1½ mile section of M-124, about to be constructed of clay-chloride stabilized gravel, was selected for a field trial of several oil-stabilized mixtures. These mixtures were laid between July 9th and 16th.

The plant used on this project was a Barber-Greene, equipped with 2 aggregate proportioners, a clay shredder, and a continuous twin pug-mill mixer. A Barber-Greene finisher was used on the road so any possible advantages of blading to dry out the mix before compaction was not obtained. This may prove to be a factor of considerable significance in the performance of the road, but from observations for the short period since construction it appears that evaporation from the compacted mixture may also produce an efficient distribution of bitumen depending, as will be shown later, on the characteristics of the bitumen.

TABLE 1

AGGREGATE GRADATION SUMMARY

	Mix Composition: Gravel plus		Aggregate Analysis: Per Cent Passing			Clay and Silt	Moisture
Date	Oil	Filler	¾" square Screen	½" round Screen	10-mesh	%	%
1938							
7/9	SC-2	Clay	97.7	81.1	49.3	14.2	7.1
7/9	SC-2	Clay	98.6	86.1	50.9	15.2	6.9
7/11	SC-2	Clay	100.0	85.6	50.9	13.5	5.6
7/12	SC-2	Sand	98.0	81.0	47.8	3.3	4.0
7/12	SC-2	Sand	97.7	85.7	56.3	3.6	4.4
7/13	SC-2	Sand	100.0	82.8	50.4	2.8	4.4
7/13	SC-2	Sand	97.3	82.6	46.2	3.1	No Test
7/15	SC-3	Clay	98.4	81.8	50.5	15.8	5.8

Oil percentages were regulated by synchronizing the aggregate feed and pump speed, the latter by the proper choice of feed and pump sprockets.

Mixtures

Two oils, SC-2 and SC-3, were used in the field tests, but because of the uncertainty as to whether the SC-3 oil would uniformly coat wet aggregate only a small part of the oil ordered was of the heavier grade.

Although a high percentage of clay filler seemed undesirable, the Engineer was of the opinion that it would be well to try one type of mixture identical with stabilized gravel with the substitution of oil for chloride. The aggregate for the clay filler mixtures contained 8% clay

and 6% silt, and the portion passing the 40-mesh sieve had a P. I. o about 8. The fine sand used as the other added filler tested as follows:

Passing No. 10 sieve 98%
Passing No. 40 sieve 82%
Passing No. 200 sieve 12%

The average mechanical analysis of daily samples of the aggregat before the oil was added are shown in Table 1. 5% passing the N 200 sieve was desired in the sand filler mixes, although the tests average but 3.2%. Moisture determinations on the clay filler mixtures average 6.3%, while the sand filler mixtures showed an average of 4.3%.

After some preliminary trials, the aggregate feed was adjusted t 1.9 cu. yd. per min., and the oil required was computed from the weight of material shown in Table 2. As in the laboratory tests the quantitie of oil are expressed as percentages of the dry weight of the aggregat

It was planned to use 1½, 2 and 3% oil with each of the tw fillers, but the 1½% SC-3 mixes were omitted because of the limite quantity of this oil. The 3% mixtures were produced mainly to observ their behavior and wearing ability as compared with the standard oi aggregate mixtures used on other projects.

TABLE 2

MATERIAL WEIGHTS

Date		Weight (damp) per load lb.	Weight (damp) per yard lb.	Moisture in Aggregate %	Wt. per cu.yd. aggregate (dry)-lb.	O lb
7/9/38	Gravel + sand filler	8800	2933	7.0	2728	
7/12/38	Gravel + sand filler	9060	3020	4.2	2893	
7/12/38	Gravel + sand filler + 3% SC-2 oil	9730	3243	4.2	3025	90.
7/13/38	Gravel + sand filler + 2% SC-2 oil	9240	3080	4.4	2895	57.
7/14/38	Gravel + sand filler + 3% SC-3 oil	9228	3076	4.4	2864	85.
7/15/38	Gravel + clay filler + 2% SC-3 oil	8540	2847	5.8	2641	52.

Material weighed after striking off level full of 3 cu. yd. truck im mediately after loading.

NOTE: Oil has been calculated as percentage of dry weight of aggregate.

The laboratory analyses of the oil mixtures are shown in Table 3 The results given in the first two columns are in error because of th loss of oil, during mixing and quartering, into the somewhat absorben laboratory table top. Subsequent samples were well mixed in tin sampl pails and duplicate samples were removed for bitumen and moisture tests

Field Observations

All mixtures were laid and compressed without difficulty, the average hickness being 6″, laid in two approximately equal layers. After drying ut, the 1½% oil-clay filler mixture became very dusty and ravelled adly so it was covered with about 2 inches of chloride stabilized gravel o preserve the riding surface. The 2% SC-2 oil-clay filler mix became uite dry with some disintegration of the surface but it was not necessary o cover the surface for several weeks. This result is consistent with he behavior which was to some extent anticipated as pointed out in he previous discussion of aggregate grading.

The desirability of using as viscous an oil as possible was proved n both 3% SC-2 oil sections. Within 2 days of warm weather, excessive lushing appeared and within a week an appreciable thickness of easily utted mat covered the entire surface. It first seemed likely that the total mount of water and oil was more than sufficient to fill the voids and vas forced to the surface by the compaction of traffic, but when no such rouble was experienced with a very similar SC-3 mix, the logical explana- ion seems to be that the SC-2 oil is too fluid to withstand the flow of apillary water to the surface during evaporation. The behavior here ioted may be significant as an indication of the manner in which the itumen is distributed. If this material were distributed in uniform films t is not particularly clear how the variation in viscosity would exhibit uch a marked influence. On the other hand if the effects produced are lue to distribution of bitumen at contact points of particles it can be eadily appreciated how a relatively small change in viscosity may prevent he bitumen from passing through the narrow throats of interstitial penings.

TABLE 3

LABORATORY ANALYSES OF OIL MIXTURES

Lab. No.	38B-955	38B-956	38B-983	38B-984	38B-986	38B-987	38B-1019	38B-1020	38B-1021	38B-1022	38B–1023	38B–1024
Oil:												
Type	SC-2	SC-2	SC-2	SC-2	SC-2	SC-2	SC-2	SC-2	SC-3	SC-3	SC–3	SC–3
%	1½	2	2	3	3	3	3	2	2	3	2	2
Filler	Clay	Clay	Clay	Clay	Clay	Sand	Sand	Sand	Sand	Sand	Clay	Clay
Screen or Sieve:	Per Cent Retained*											
¼″ Sq.	0	0	0	0	0	0	0	0	0	0	0	1.6
½″ Rd.	19.2	16.6	15.0	17.6	15.0	17.0	17.2	19.2	11.3	19.2	18.1	17.1
¼″ Rd.	17.0	20.1	18.0	18.1	16.7	15.6	16.0	15.6	18.0	18.4	16.0	14.6
0	13.0	15.5	15.5	16.8	16.1	14.5	13.9	13.5	15.4	14.8	13.2	13.9
0	19.9	24.6	23.9	24.5	25.0	23.9	23.9	24.5	27.5	22.9	20.1	22.3
0	10.7	10.1	12.5	10.1	12.0	16.1	19.2	19.7	19.8	15.5	10.9	12.9
00	3.3	2.8	3.7	2.6	3.3	4.4	4.0	3.6	3.7	3.2	3.7	3.5
P200	16.2	9.2	9.6	7.4	9.0	5.7	2.3	2.0	2.1	2.3	16.1	12.6
Bitumen	0.7	1.1	1.8	2.9	2.9	2.8	3.5	1.9	2.2	3.7	1.9	1.5
Moisture	6.2	6.0	5.5	5.6	6.1	5.2	4.5	4.6	5.2	4.1	5.6	5.5

*Mixture analyses given on moisture free basis.

After 2 weeks of observation, during which the impervious mat seemed to prevent further evaporation of water, these two sections were aerated with blades to a depth of about 4 inches and were then bladed to uniform section and contour. After the evaporation of the excess water no bleeding occurred.

As previously mentioned, the SC-2 oil was used primarily because of the doubt that SC-3 would mix uniformly with wet aggregate. This fear proved ungrounded and the distribution of the heavier oil seemed equally as good as the SC-2 oil. One additional precaution, found necessary with the SC-3 oil, was that it be heated until quite fliud, because when too viscous the oil rate of flow decreased appreciably for the same pump speed.

The 2% SC-3 oil sections were definitely superior to any SC-2 mixtures. There was much more ravelling in the clay filler mixtures than in the sand filler mixtures. This seemed to be due mainly to the disintegration of clay pockets. These mixtures were left open to traffic for a six weeks period for observation. Normally they should be sealed after the evaporation of excess water has taken place, which is about a week in warm weather.

The 2% SC-3 mixture was surprisingly cohesive for such a low oil content and it appears entirely probable that, with sufficient moisture to aid mixing and to eliminate segregation during the laying operations, and with proper drying conditions, 1½% of SC-3 may be ample to provide a water repelling base for bituminous surfaces. It should be pointed out that while the tests performed and observations of the experimental project indicate very favorable performance, the long time behavior of the mixtures is yet to be studied.

After 6 weeks of traffic the entire test section was double seal coated with MC-5 cut-back asphalt and chips, as was the adjoining chloride stabilized base (329 feet west of, and 1400 ft. east of, the oil stabilized sections). This road will be under observation, especially during the coming winter and spring seasons.

CAPILLARY WATER ABSORPTION DATA

Curve No.	Bitumen	Mixture Composition	Method of Drying	Absorption Days	Absorption %-Dry Basis
1	SC-2 oil	Gravel	No Moisture Present	7	7.2
		Gravel+1% oil	No Moisture Present	7	5.7
		Gravel+2% oil	No Moisture Present	7	1.2
		Gravel+3% oil	No Moisture Present	7	0.5
2	SC-2 oil	Gravel+2% water	Windrowed for 2 days on laboratory table- turning over several times daily.	7	7.8
		Gravel+2% water+½% oil		7	0.8
		Gravel+2% water+1% oil		7	0.7
		Gravel+2% water+1½% oil		7	0.4
		Gravel+2% water+2% oil		7	0.5
3	SC-2 oil	Gravel+2% water	Not dried	7	6.2
		Gravel+2% water+½% oil	Not dried	7	4.0
		Gravel+2% water+1% oil	Not dried	7	2.0
		Gravel+2% water+2% oil	Not dried	7	0.6
4	SC-2 oil	Gravel+2% water	Dried to constant weight in oven at 60° C.	7	7.2
		Gravel+2% water+½% oil		7	0.5
		Gravel+2% water+1% oil		7	0.5
		Gravel+2% water+2% oil		7	0.4
		Gravel+2% water+3% oil		7	0.3
5	SC-2 oil	Gravel	Mixed dry and heated for 2 days at 60° C.	7	7.2
		Gravel+½% oil		7	2.2
		Gravel+1% oil		7	0.5
		Gravel+2% oli		7	0.3
6	SC-2 oil	Gravel+15% clay+5% H2 O	Dried to constant weight in oven at 60° C.	7	7.7
		ditto+1% oil		7	1.4
		ditto+2% oil		7	0.5
		ditto+3% oil		7	0.2
		Gravel+20% clay+7% H2 O+3% oil		7	0.9
7	SC-2 oil	Gravel+4% water	Windrowed for 2 days on laboratory table- turning over several times daily.	7	7.8
		Gravel+4% water+½% oil		7	0.8
		Gravel+4% water+1% oil		7	0.5
		Gravel+4% water+2% oil		7	0.4
		Gravel+4% water+3% oil		7	0.4
8	SC-3 oil	Gravel+2% water	Ditto Above	7	7.8
		Gravel+2% water+½% oil		7	0.7
		Gravel+2% water+1% oil		7	0.5
		Gravel+2% water+2% oil		7	0.4
		Gravel+2% water+3% oil		7	0.4
9	Soil Stabilization Emulsion	Gravel+4% water	Ditto Above	7	7.8
		Gravel+4% water+½% Emul.		7	1.0
		Gravel+4% water+1% Emul.		7	0.6
		Gravel+4% water+2% Emul.		7	0.5
		Gravel+4% water+3% Emul.		7	0.5
10	TM-2 Tar	Gravel+4% water	Ditto Above	7	7.8
		Gravel+4% water+½% tar		7	1.6
		Gravel+4% water+1% tar		7	0.8
		Gravel+4% water+2% tar		7	0.5
		Gravel+4% water+3% tar		7	0.4

ANALYSES OF SLOW-CURING OILS

	SC-2	SC-3
Specific Gravity, 25°/25° C.	0.950	0.963
Water, per cent	0	0.2
Flash Point, Cleveland Open Cup, °C.	199	224
Viscosity, Saybolt Furol at 50° C., sec.	148	
Viscosity, Saybolt Furol at 60° C., sec.		173
Heterogeneity Test	Negative	Negative
Solubility in CCl_4, per cent	99.86	99.64
Loss on Heating, 50 g., 5 hr., 163° C., per cent	0.56	

Distillation Tests—		
1. 0° C. to 190° C.	0.0	0.0
2. 0° C. to 222° C.	0.0	0.0
3. 0° C. to 315.5° C.	0.0	0.0
4. 0° C. to 360° C.	Trace	0.0
Tests on 100 Penetration Residue—		
Total Residue, per cent	64.8	73.0
Ductility at 25° C., 5 cm. per min.	105	87
Ductility at 4° C., 5 cm. per min.	11	10.5
Actual Penetration of Residue at 25° C.	97	102

ANALYSIS OF TAR TM-2

Water, per cent	Trace
Specific Gravity at 25°/25° C.	1.151
Specific Viscosity, 50° C.	30.5
Distillation Test— (on water free material)	
1. 0 - 170° C.	0.
2. 0 - 270° C.	17.3
3. 0 - 300° C.	25.1
Softening Point of Distillation Residue, °C.	46.1
Total Bitumen Soluble in CS_2, per cent	93.73

SPECIFICATION FOR SOIL STABILIZATION EMULSION

Viscosity—Saybolt Furol 25° C. sec.	40 to 100
Specific Gravity, 25°/25° C.	Not less than 1.00
Residue at 163° C. 3 hr. 50 g.	55% to 60%
Mixing Test	Not more than 5% broken
Settlement, 10 days	Not more than 3
Sieve Test	Not more than 0.10%
Demulsibility, 50 ml. 0.1 N $CaCl_2$, per cent	Not more than 2%
Dehydration, 100° F., 96 hr.	Not less than 60%
Tests on Petroleum Asphalt before emulsification—	
Penetration, 25° C., 100 g., 5 sec.	50 to 100
Solubility in carbon disulfide	Not less than 99%
Loss at 163° C., 50 g., 5 hr.	Not more than 1.0%
Ductility at 25° C., cm.	Not less than 120 cms.

Present Practice of Bituminous Road Construction in Alabama

By

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The developments in the design and construction of bituminous pavements in this state have advanced sufficiently to warrant a complete review of the present practice.

Our bituminous pavement is designed on the basis of the following fundamental principles which were derived from practical research obtained from observation and study of this type of construction over a period of years and covering more than one thousand miles of surfacing.

1. Through compaction of the roadbed, sub-base or subgrade.

2. Construction of a base course made of natural selected soils to provide a flexible foundation that will supplement the bearing power of the roadbed soil, so that the finished course will support the weight of the vehicles. The base course is built of materials which have low volume change, sufficient adhesion and satisfactory soil characteristics and stability to yield but not be displaced by the vehicular loads and the destructive actions of nature.

3. A wearing surface that is waterproof as nearly as possible so that it will prevent surface water from entering the base course, is flexible yet rigid enough to prevent shoving, and has a non-skid finished surface that is safe at all times.

4. The work must be done under rigid engineering control to assure the best construction possible considering conditions encountered. With these fundamentals in mind each new project selected for paving is studied to determine the changes necessary in the alignment, cross-section, and grades in order to make the road as safe as possible. This study is based on traffic, local conditions and a due regard for the allotment appropriated for the improvements. Due to the large mileage of roads programmed for paving each year, appropriations are not sufficient on many projects, to properly regrade the roadway to conform to the best present practice of alignment, cross-section, and gradient at this time.

In connection with the preliminary study a soil survey is made of the entire road. This consists of sampling and analyzing the natural soil in the roadbed, locating, investigating, sampling, and analysing the materials to be used for the base course. After the soils have been analyzed, we determine the thickness of the base course to be used based on the behavior of such soil on other projects that have been completed; the type of base course material found in the vicinity, and traffic conditions. For example, in one County where we have plastic A-7 soil, we use a minimum of one foot of selected sandy loam material as a subgrade topping before placing a base course of sand-clay eight inches minimum thickness. While in another locality where A-2 friable soil is general, a six-inch gravel base will be sufficient without the necessity of any subgrade topping material. All such decisions are based on the best judgment of the Engineers designated to make such studies. We do not have any laboratory test to cover this important phase of the design other than the regularly used mechanical gradation and soil constant tests.

We have previously discussed the general method of our work, hence with our added experience in such construction a review of some of the details will be given.

Four years of this type of construction indicates the necessity of building a satisfactory statewide organization which is capable of carrying on a large program of such work (this year we are building more than 800 miles of surface treatment). This required a lot of hard work, study, and personal training of engineers and inspectors so that they not only understand the work, but can cooperate with the contractors to devise means to do the work better and at a lower unit cost. While the engineering department was going through its development stages we found this condition to be even harder on the contractors side as it was necessary to train so many superintendents, foremen and operators of equipment. With both organizations better trained we find ourselves getting work at almost 60% of the unit costs of 4 years ago at the same time considerable improvement in the quality of the work.

Our engineers now watch the subgrade very closely so that it is about a thing of the past to visit a project and find the contractor placing base course material on a soft, spongy, loose or extremely dusty subgrade. Next, we are confronted with the problem of proper base construction. Our present practice is to place all base courses in two layers. Each layer is dumped directly on the compacted subgrade which has first been brought to the proper cross-section by machining if on a newly constructed grade or by the use of levelling courses if placed on an old roadbed. We do not use form boards for guides. The engineering control exercised by the use of grade stakes, strings and templets and the advancements in machinery design and operation is such that we can finish the base course to a satisfactory riding grade and cross-section with the motor patrol or other similar equipment. Each course is placed and spread and then mixed. In the mixing we have encountered many problems. All the mixing is done on the roadway by windrowing, plowing and harrowing. Different materials handle differently; sometimes we have to windrow the materials; sometimes it is necessary to wet the material before mixing; or we have to mix the material while

in its natural state of moisture and then wet the material and mix again. While all this sounds complicated we have developed the art so that a contractor can place and mix as much as 3000 cu. yds. of base material in a day. One of the biggest problems is to shape the base course before priming. At first we just machined the road and then primed same. This was not satisfactory because the road was not always to the proper shape and cross section. Then we started to scarify the top of all base courses and reshape the loose material to the proper section, then recompact same. This was not entirely satisfactory either because many of the projects were not sufficiently recompacted. Also, in reshaping there developed a lot of sandy spots that would ravel after the prime was placed. Our present practice based mostly on the improvement in the art of doing the work is to start compaction as soon as the base course is mixed. This is done by wetting and proper machining. On many roads we do not let the base course get dry during the compaction period. By the careful use of the templet in connection with the machining operations we can get the base course in proper shape and hold same in that shape during the compaction period so that no additional work is needed before the prime is placed. When the base is on the sandy side, one must be careful in machining so as to prevent segregation and loss of the clay binder. This method is not used, however, on many of our top-soil and sand-clay base courses. For such bases generally we do not have any material above the 10-mesh sieve; hence, after every rain or machining the material has a tedency to laminate so that if the base is not reworked it will scab or flake off. When this condition occurs, we find it necessary to first scarify the base about 3 inches, then remix it while dry and reshape same. Then we place artificial traffic (either trucks or pneumatic-tired rollers) to compact same. After a reasonable amount of such traffic the base is wetted and compaction continued, until properly compacted. The machining and shaping is carried on at the same time until the base is ready for priming. This is one of the hardest operations we have and unless the work is done by experienced men it will not be very successful. All the above is mentioned so as to emphasize the fact that after the fundamentals are properly worked out, the work of construction must be developed as an art until construction details are properly cared for. We still have many construction problems to solve in the way of getting more uniform base course thickness, better riding surfaces, and better compaction; but we are always working on the problem.

I have devoted most of this discussion to the foundation work because that is the important part of the successful construction of low cost pavement. Our top consists of first cleaning and applying a prime coat. This application is applied at the rate of approximately 0.3 gallon per square yard. With improvements in the compaction of the base, we have found that we are able to use less material in this application.

This year in our inverted penetration or surface treatment course, we have used smaller aggregate on many of the projects. We have changed from a minimum of .4 gallon hot application covered with ½ cubic foot of 1 inch to ½ inch aggregate to a minimum of .35 gallon covered with .45 cubic foot of ¾ inch to ⅜ inch aggregate. All figures are rates

per square yard. The hot application is usually 150 to 200 penetration asphalt; sometimes RT 12 tar. The aggregate is either round gravel, crushed slag or stone. The use of the smaller aggregate reduces the cost about 1½¢ per square yard. Also, it is easier to get a satisfactory cover with the smaller aggregate.

In the past our wearing course application has been a drag seal made with .42 gallons of cut-back asphalt or tar in two applications and dragged after applying ⅜ inch to 16-mesh aggregate. While this was fairly satisfactory we found that this construction was such as to make it difficult to enable us to get good results, especially in the fall. Also, we felt like it was not worth the cost—generally 8 cents to 10 cents per square yard. Our present practice is to finish the surface treatment by an application of approximately 35 lbs. per square yard of cold plant mix material. This application fills the larger voids of the surface treatment and provides a more densely graded wearing surface that does not wear tires so much and, furthermore, prevents wear of the surface treatment aggregate. The plant mix seal retards the weathering of the bituminous materials because it does not have the large surface area to weather as is the case in the very open type wearing surfaces.

This year most of the spreading and finishing of the plant mix material has been done with self-propelled spreading and finishing machines. Some work has been spread by dumping on the road from the truck and spreading with motor patrols. Of course, the machine finishing is far superior as it leave a smoother surface and elmiinates so much of the segregation of the mixed material.

A large part of the plant mix material used this year has been obtained from commercial mixing plants. Such mixtures are shipped by rail to the project. Here our problem is to use the correct liquefier and the proper amount. As long as the weather is constant, we do not have any trouble, but when the weather changes our trouble begins, because if we use too much or too heavy a grade of liquefier, the mixture does not set up quick enough to prevent the material from tracking under traffic, particularly mule drawn wagons. The other difficulty is that too much liquefier, will soften the bitumen in the surface treatment and cause it to roll or wave.

The specifications for the plant mix seal is as follows:

Passing ½ inch sieve	100%
Passing ⅜ inch sieve	85 to 100%
Passing No. 4 sieve	20 to 65%
Passing No. 10 sieve	20 to 65%
Passing No. 40 sieve	5 to 35%
Passing No. 200 sieve	0 to 10%
Bitumen and liquefier	5 to 8%

When the mix is made for shipment we use 85 to 100 penetration asphalt and a liquefier. Sure mixes are made with dried and cooled aggregate only. When the mix is made on the project, if we use an asphalt plant we use 150 to 200 penetration asphalt without the liquefier. This is a semi-hot mix. If we do not have an asphalt plant we do not dry

he aggregate and use an RC-3, 4, MC-3, or MC-4. Tar is used in lieu of sphalt in all bituminous work on some projects. While we have obtained good results with wet aggregates its success depends too much on obtaining warm and dry weather for the proper curing after the material is placed on the road.

In conclusion, I might state the construction of the pavement is about as much an art as that of the base and unless one has the proper trained personnel, it is most difficult to obtain satisfactory results.

Sand Bituminous Low Cost Road Construction in Florida

By

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The development of mixed-in-place sand bituminous road mix pavement in Florida was started about nine years ago in order to provide a low cost type of road surface for the section of the western part of the State bordering on the Gulf of Mexico, where the soil is of a sandy nature and where lack of railroad facilities and local material made the cost of constructing a higher type pavement practically prohibitive. However, during the past few years projects incorporating this type pavement have been constructed in almost all sections of the State.

Results obtained in Florida in the construction of this type pavement indicate that it can be constructed with good results on any road where the existing road material is sand, or a mixture of sand and stone, gravel or shell, in the proper proportions to form a dense mixture. It has been found that the presence of more than about 10 per cent of clay, loam, silt or organic matter tends to cause the material to ball up when mixing with the bituminous material, thus preventing a uniform mix. A road, to be suitable for the construction of this type pavement, must be adequately drained. A well drained subgrade is, of course, essential to any type pavement, but it is even more essential for the sand-bituminous road mix.

Sand-bituminous road mix as constructed in Florida is a bituminous mixture composed of the existing road material mixed thoroughly with asphaltic oil, coal tar or petroleum tar, in which all mixing is done mechanically on the road.

The first step taken when this type pavement is to be constructed on a project, after adequate drainage has been provided, is to make a survey of the existing road material by taking representative samples from the surface to a depth of about 10 inches. The samples are then analyzed for percentage of clay, loam, silt and organic matter; a sieve analysis is made to determine gradation, and tests are made to determine the stability of the material. As has been stated before, the presence of more than about 10 per cent of clay, loam, silt or organic matter is objectionable and should be remedied by either adding enough suitable material so that the resultant blend will be satisfactory, or by removing the unsuitable material and backfilling with suitable material. The gradation of the material is not so important except insofar as it affects the stability; however, a well-graded coarse sand is preferable. The finer the sand in size the more bituminous material will be required due to the

greater surface area of the finer material. If material can be obtained locally and at low cost, it is sometimes more economical to add coarse material to the finer existing material than to use the extra bituminous material required for the finer material by itself. However, except for economy, the addition of material to correct faulty gradation is seldom justified. In order to secure a satisfactory surface of this type, it is extremely important that the existing road material be fairly stable. From results obtained in Florida it has been determined that the existing road material should have a bearing value of not less than 25 pounds per square inch. This bearing value is determined by our Testing Division by means of a machine which they have devised and which is patterned after the U. S. Bureau of Public Roads sub-soil bearing value machine, and which consists of a metal cup 3 inches deep and 3 inches in diameter, in which material to be tested is placed and subjected to a pressure of 1200 pounds; then a circular disc equivalent to one square inch in area is placed on the surface of the material and through a lever system a constantly increasing load is applied until the pressure on the bearing plate upsets the stability of the material and the plate goes down into the material. The load at failure is recorded as the bearing value. The material before being tested is oven dryed; all lumps broken up, and to 600 grams of the dried material is added 10.5 cc. of water which is well mixed into the material. In case the bearing value of the existing road material is found, by the above test, to be less than 25 pounds per square inch, a stabilizing material should be added to bring the resultant blend up to a bearing value of 25 pounds per square inch or more.

The State Road Department of Florida has experimented with several grades of asphaltic oil and tar and has secured the best results from material meeting the following specifications:

1. **Asphaltic Oil (Cut-back Asphalt)**

This material shall consist of an asphaltic cement base cut back with naphtha.

The asphaltic cement base shall be homogeneous, free from water, and shall not foam when heated to 347° F. blown or oxidized asphalt will not be accepted. It shall meet the following requirements:

(1) Penetration at 77° F., 100 g., 5 secs.—Minimum 85, Maximum 100.
(2) Flash Point, Open Cup, Degrees F.—Minimum 347°.
(3) Loss 325.4° F., 50 g., 5 hurs.—Maximum 1%.
(4) Ductility at 77° F., cms.—Minimum 100.
(5) Bitumen Soluble in CS_2—Minimum 99.5%.
(6) Bitumen Soluble in CCl_4—Minimum 99%.
(7) Residue Penetration at 77° F.—100 g.—5 Secs.
Per cent of original—Minimum 60.

The cut back asphalt shall conform to the following requirements:

(1) Viscosity, Furol, at 122° F.—Minimum 70 to Maximum 140.

(2) Distillation A.A.S.H.O. T-78, with the following exceptions: Distillation shall be stopped at 680° F., and the entire residue

shall be immediately poured into standard penetration containers and allowed to cool for further tests.
Water (per cent by volume)—0 to 1.
Distillate (per cent by volume)
To 302° F.—Minimum 0 to Maximum 5.
To 437° F.—Minimum 20 to Maximum 30.
To 600° F.—Minimum 26 to Maximum 38.
To 680° F.—Maximum 38.

(3) Tests on residue from above distillation:
Penetration at 77° F., 100 g.—5 secs.—85 to 100.
Solubility in CS_2 (per cent)—99.5.
Ductility at 77° F., cms.—60.

2. **Coal Tar.**

The tar shall be homogeneous and shall consist of a heavy coal tar base, containing pitch oil, which is fluxed to the specified viscosity with water gas tar or distillates thereof, or light coal tar distillates. The volume of pitch oil shall be from 3 to 15 per cent of the volume of coal tar base, as directed by the Engineer. The tar shall also conform to the following requirements:

(1) Engler Specific Viscosity at 140° F.—Minimum 12—Maximum 32 (shall not vary more than 3 points plus or minus from specific viscosity designated by the Engineer).
(2) Per cent of water—Maximum 1%.
(3) Total Distillate by wt. to 338° F.—Minimum 0—Maximum 2.
(4) Total Distillate by wt. to 455° F.—Minimum 0—Maximum 12.
(5) Total Distillate by wt. to 518° F.—Minimum 0—Maximum 18.
(6) Total Distillate by wt. to 572° F.—Maximum 30.
(7) Bitumen per cent soluble in CS_2—Minimum 88%.

3. **Petroleum Tar.**

The tar shall be homogeneous and shall consist of not less than 60 per cent of refined petroleum tar base, fluxed to the specified viscosity with a tar material (liquid at 60° F.). It shall have the characteristics of tar and when combined in proper proportion with road material from the proposed sand-bituminous road mix project, and air cured, the mixture shall show a tendency to set within 6 hours, have a firm set in 4 days and hard set in 7 days. It shall also conform to the following requirements:

(1) Engler Specific Viscosity at 158° F.—Minimum 22—Maximum 32 (shall not vary more than 3 points plus or minus from specific viscosity designated by the Engineer).
(2) Per cent of Water—Maximum 1%.
(3) Total Distillate by wt. to 338° F.—Minimum 0—Maximum 5.
(4) Total Distillate by wt. to 572° F.—Minimum 15—Maximum 40.
(5) Bitumen per cent soluble in CS_2—Minimum 88%.
(6) Specific Gravity—Minimum at 77°/77° F. —1.10.

The above specifications for asphaltic oil (cut back asphalt) are varied slightly when conditions warrant by changing the asphaltic content

and per cent of naphtha and the consistency of the base asphalt. Instances have been encountered where the percentage of clay, loam, silt, etc., was a little high and it was not practical to lower this percentage, in which instances the penetration of the base asphalt was changed to be 100 to 140 instead of 85 to 100, giving a softer grade of material and one which mixed more readily without balling up. However, it is best to hold to the 85 to 100 penetration for the base asphalt unless such cases are encountered.

The major items of equipment forming a unit for construction of this type pavement are:

(1) One pressure distributor, or trailer tanks with trailer type distributor. The trailer type is preferable.
(2) Not less than one grader with 12-foot blade, and not less than than 12-inch rims. Maximum weight of grader 7,500 pounds.
(3) Three disc harrows—22-inch disc of an approved type for proper mixing.
(4) One 3-furrow gang plow—14-inch bottoms, capable of plowing to a depth of 10 inches.
(5) One 3-section spring tooth harrow with alfalfa teeth.
(6) Sufficient tractors of the proper size and horsepower to handle the equipment used.
(7) One retread mixing machine of an approved type.
(8) One 4- to 6-ton tandem roller.
(9) If the temperature is fairly low at time of mixing, an approved heater for heating the blade of the grader is sometimes necessary.

The construction of this type pavement should at all times be under the direct supervision of someone who has had experience on work of a similar nature with the same type bituminous material.

The method of construction used by the State Road Department of Florida in constructing a sand-bituminous road mix pavement with a finished thickness of 6 inches, is as follows:

The existing road material is plowed to a depth of 10 inches and to a width of 2 feet outside the proposed edge of the finished pavement. The subgrade is then dragged with the spring tooth harrow and all roots and foreign matter removed as far as practicable. All stones or pieces of rock that will not pass a 2-inch ring are removed to a depth of 10 inches and for a width of 2 feet outside the edge of the proposed finished pavement. The road material is then machined with the 12-foot grader until a uniform grade and cross section is obtained.

The amount of bituminous material required varies with the character and gradation of the material with which it is to be mixed. This type of mix follows the theory of all asphalt mixtures in which the quantity of bituminous material shall be sufficient to entirely coat each particle of the mineral aggregate, and a slight excess to fill voids and produce durability. The method of determining the quantity of bituminous material required is a combination laboratory investigation and trial mix on the job. After the road material has been sampled and analyzed for

quality and gradation in the laboratory, small batches are made up and from the results of this study the approximate quantity of bituminous material necessary for good results is determined. Just enough bituminous material should be applied so that after thorough mixing the resultant mixture will be homogeneous and will have a black, glossy appearance.

After the existing road material has been cleaned and shaped, and any unsuitable material has been replaced with suitable material, or enough suitable material has been added to remedy any defects in the existing material, the bituminous material is applied by means of a mechanical pressure distributor equipped with spray bars capable of spraying the entire width of the proposed pavement. A pressure varying between 20 and 75 pounds per square inch is maintained on the distributor. If coal tar or petroleum tar is being used, the temperature of the material is held above 90° F. while being applied. If cut back asphalt is being used, it is preferable that it be applied at air temperature. If heating is necessary for proper distribution, the temperature of the material is not allowed to exceed 120° F. The bituminous material is only applied when the existing road material is comparatively dry. No bituminous material is applied when the air temperature is less than 60° F. in the shade.

Each application of the bituminous material consists of approximately 0.35 gallon per square yard. Mixing is done with the disc harrows, the 3-furrow gang plow and the 12-foot grader. Each application of bituminous material is followed by continuous mixing with the harrows. After approximately 2 gallons of bituminous material per square yard has been applied, the entire width of the pavement is plowed with the gang plow to obtain uniform depth and mixing with the harrows is continued. Material in place is then windrowed to a depth of 3 inches with the 12-foot grader, from center to outside edges of pavement. After windrowing, application of bituminous material is continued for the full width of the pavement and the material is mixed with the harrows and rolled from side to side with the 12-foot grader until a homogeneous mixture of the required richness and depth has been obtained.

Material that has been windrowed outside of edges is then pulled in and spread with the 12-foot grader to a uniform surface. After material has been spread the edges and quarters are plowed to obtain uniform depth.

Application of bituminous material is then continued until a black, glossy surface has been obtained. In mixing the upper 3 inches, which is to be the wearing surface, harrows are used and during the last 3 or 4 applications of bituminous material the mixing machine is used continuously set to cut from 2½ to 3 inches. In conjunction with the mixing machine, the 3-section spring tooth harrow is used to further perfect the mix. After a sufficient quantity of bituminous material has been applied, mixing is continued until the materials are thoroughly mixed.

After mixing is completed the surface is bladed to the desired grade and cross section with the 12-foot grader, and all irregularities in the surface corrected, properly mixed material being added if required.

All of the above described operations are done as soon as possible after application of the bituminous material is started on a section of

road, in order that mixing can be completed before too much of the naphtha or other fluxing material is lost by volatilization.

After the pavement has partially set, it is rolled with the 4- to 6-ton tandem roller equipped with scrapers and oil sacks on the wheels to prevent picking up of the material. This initial rolling is done longitudinally, beginning at the edges and proceeding toward the center. After the pavement has set several days more, but before it has set hard enough to prevent rolling out irregularities and tractor and other marks, final rolling is done both longitudinally and diagonally until a uniform compaction and smoothness has been obtained, and all tractor and other marks are eliminated.

If coal tar is used, after final rolling is completed, and about one month after mixing operations are completed, the finished surface is treated with a seal coat of 0.2 gallon per square yard of coal tar of the same quality as used in the mix, and covered lightly with sand to prevent picking up by traffic.

If petroleum tar is used, a seal coat is applied as above described, except that the material used for the seal coat is 0.25 to 0.35 gallon per square yard of cut back asphalt. If considered necessary, this seal treatment using cut back asphalt may be applied where the material used in the mix was cut back asphalt. However, a seal coat is not generally required where cut back asphalt is used in the mix.

After rolling has been completed, the edges of the pavement are trimmed to a straight line. Suitable material cut from the edges is used in constructing driveways, turnouts, and so forth.

Traffic is not allowed to use the pavement until it has set hard enough to eliminate rutting and cracking.

The materials and construction methods discussed in the foregoing are those which have been used to date in the construction of this type pavement in the State of Florida.

We have made investigations relative to the use of a flux, powdered hard asphalt and asphaltic cement as a bituminous material for use in this type pavement, and in the future expect to use this as an alternate to the three other materials, the specifications for which are given in the foregoing. The specifications for the flux, powdered hard asphalt and asphaltic cement which we intend to use are as follows:

The flux oil shall conform to the following requirements:

(1) It must be homogeneous and free from water.
(2) Specific gravity at 77°/77° F., not less than 0.97.
(3) Viscosity, Furol, at 122° F., 600 to 800 seconds.
(4) Loss on heat, 5 hrs., 50 grams, at 325° F., not more than 5%.
(5) It must be capable of complete amalgamation with the hard asphalt.

The hard asphalt shall conform to the following requirements:

(1) Penetration at 77° F., 100 G.—5 seconds—maximum 3.
(2) Penetration at 115° F., 100 G.—5 seconds—maximum 8.

(3) Bitumen Soluble in CS_2—minimum 99%.
(4) Bitumen Soluble in CCl_4—minimum 95%.
(5) Softening point, R & B, Degree F—minimum 200.
(6) It shall be reduced to powder in an impact mill, and as used, not less than 50% shall pass an 80-mesh sieve and 95% shall pass a 20-mesh sieve.

The hard asphalt when combined with the flux oil in a proportion between 50% and 70% of flux at a temperature of 325° F., shall produce an asphaltic cement of 150 penetration at 77° F., and in the manufacture of the pavement they shall be mixed in such proportion as will produce, when completely amalgamated, an asphaltic cement of the penetration required by the Engineer and conforming to the following requirements:

(1) Specific Gravity at 77° F.—Minimum 1.02.
(2) Penetration at 77° F., 100 G., 5 seconds—Minimum 80—Maximum 200. (The penetration shall not vary over 5 points either way from the penetration designated by the Engineer).
(3) Flash Point, Open Cup, Degrees F.—Minimum 347.
(4) Ductility at 77° F. cms.—Minimum 100.
(5) Bitumen Soluble in CCl_4—Minimum 95%.
(6) Residue Penetration at 77° F., 100 G.—5 secs., per cent of original—Minimum 50.

We have also made investigations relative to the use of a traveling mixing plant in the construction of this type pavement, and in the future will make mixing with a traveling mixing plant optional to mixing in place. When a traveling mixing plant is used, a distributor will not be necessary except where a seal coat is required.

In mixing with a traveling mixing plant, the existing road material is thrown up in windrows suitable for the machine to pick up. The windrows are sized and shaped to secure uniform feed. The plant must travel at a uniform rate of speed and elevate the material cleanly without gouging or otherwise injuring the subgrade. The bituminous material is added in the mixing hopper in the proper proportions, except when flux and powdered hard asphalt is used, in which case the bituminous materials are added to the existing road material in the pug mill.

After the material has been mixed by the traveling mixing machine, the material is worked on the road with harrows and graders until the mixture has aerated and cured sufficiently for final finishing and compaction to be done.

In Florida the greater part of this type pavement has been constructed using cut back asphalt as the bituminous material. However, we have constructed a considerable amount using tar and have secured good results from this material. Tar is much slower setting up than cut back asphalt and in using tar it is better to use a somewhat lean mix, thus requiring considerably more harrowing and blading to secure a good mix. Also, if the bearing value of the existing road material is too high and tar is used, the finished product has a tendency to crack.

To date we have constructed over 400 miles of this type pavement

in Florida. The cost has varied from a minimum of around 40 cents per square yard to a maximum of about one dollar per square yard, averaging about 65 cents per square yard. These costs are for the surfacing only and do not include any excavation costs. The variation in cost is due to the variation in quantity of bituminous material required and amount of mixing required. The quantity of bituminous material required has varied from a minimum of about 3 gallons per square yard to a maximum of about 7 gallons per square yard, averaging about 5 gallons per square yard, depending upon the character and gradation of the existing road material.

The majority of this type pavement has been constructed on secondary roads where traffic is light, although we have some on roads which carry comparatively heavy traffic and it is standing up well; however, it must be remembered that this type pavement cannot be classed as a high type pavement and its construction is recommended only where a good surfaced road is needed but where a low traffic density and other considerations make it impractical to construct a higher type pavement.

Developments in the Construction of Bituminous Surface Courses in Wisconsin

By

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State Highway Commission of Wisconsin

Early in 1937 the policy had been established that there would be no dusty roads on the state trunk highway system in Wisconsin. While the state had a large mileage of both Portland Cement concrete and bituminous surfaced roads, there was a considerable mileage on the state trunk highway system which still had received no dustproofing treatment. On most of these roads there was a varying amount of gravel or crushed stone and on some of them there was no road metal at all, particularly in the more sandy regions of the state. As it was impossible to give all of these roads an adequate surface course of gravel or crushed stone, it became necessary in quite a few instances to treat the material as found on the existing surface with a dustproofing medium. On most of these roads, therefore, a treatment was given using a slow curing oil, most of which was required to show a negative spot test. By the end of the summer all of the highways in Wisconsin had received such a treatment so that there were no dusty roads on the state trunk highway system in Wisconsin. This policy was extended to the detours around construction jobs and in the setup for the construction project arrangements were made for the financing of the oiling of the detours.

On any construction work where gravel or crushed stone is contemplated, as soon as such surfacing is placed, if the weather is still seasonable, the same is given an application of oil to dustproof it and to conserve the surface. A light mat only is provided to accomplish the above and further mat construction is deferred for a year or so, so that the grade can obtain settlement and weak places in the grade can be strengthened as well as to provide an opportunity for observation of subgrade conditions which were not apparent during the time the grading operations were being done so that these can be corrected prior to the construction of the final surface course.

In the matter of subgrade correction, Wisconsin being a state which has been glaciated over most of its area, making soils differentials quite

prevalent, more and more attention is being paid to the recognition or location of such soils differentials during the time of the original survey and during the time grading operations are in progress, and provisions are made to take measures either in the original plans or as extra work to correct conditions where subgrade failures might occur. Most of the work done in this regard consists of the removal of the offending soils and the backfillings of the places thus excavated with suitable materials. In quite a number of instances also the releasing of so-called perched water tables by cutting through the dikes of impervious soils existent in pervious soils will create a stable condition in the subgrade. In a number of instances also, sand lifts about 12 inches in depth over existing grades have proved effective in the elimination of bad subgrade conditions.

In new construction work a stable base about 4 to 6 inches in compacted depth, depending upon the type of highway, is attempted to be developed using aggregates that fall within a band or range as shown in a previous volume of the proceedings of the Montana Bituminous Conference, and a similar gradation is set up for the bituminous mat placed on this base using a slow curing oil binder. The mat is constructed by road mix methods using blade graders and equipment of a similar nature. The percentage of bitumen is determined by formula; however, in quite a number of instances it is necessary to modify the percentage thus determined by visual inspection of the appearance of the mix. Studies are being made on aggregate gradation on complete road mix jobs as well as the bitumen content on these mixes in an effort to determine the degree of uniformity of gradation and dispersion of bitumen in mixes placed by the road mix method. At the present time, however, these studies have not been carried far enough along for us to be able to draw a definite conclusion.

A few plant mix jobs using stationary plants with dryers and weight proportionment of materials have also been constructed in the state during the past two years. In most of these plant mix jobs, however, tar is used for a binder. Based on experience gained on one job last year where aggregates were used in which a percentage of the maximum size was retained on a 1″ sieve, the department has this year reduced the maximum size so that all of the material will pass a 1″ sieve. The difficulty experienced using the larger size aggregates and placing the mat with a self-propelled spreading and finishing machine was that some segregation took place causing more or less stony pockets in which a uniform amount of compaction could not readily be obtained under rolling, causing some slight uneveness in the surface. By reducing the maximum size, this difficulty has been to a large extent eliminated. The gradation has now been set up as follows:

Retained 1″ sieve	0%
Retained ¾″ sieve	10-20%
Retained No. 4 sieve	35-55%
Retained No. 30 sieve	65-80%
Passing No. 200 sieve	0-10%

These graduations are the master ranges and are the maximum and minimum for all cases. The contractor is required to set up a closer con-

trol appropriate to the job materials and in several instances he has set up a control as being the midpoint between the master ranges given and so far the experiences indicate that he can control the gradations within a small percentage one way or the other of the middle of the range. The plant provides for two-bin separation of the aggregates which are separated on the No. 4 sieve and are stored in separate compartments and recombined by weight in the proper proportions in the mixer. The bitumen content thus far has been between 4 and 4½% by weight. Extraction tests indicate that these percentages and gradations are kept within very close cognizance in the finished product.

In the matter of controlling densities in rolling of the mat, density determinations are made in the laboratory using samples of the aggregate as produced and test samples are taken from the finished mat and density determinations made thereon. When laboratory densities are approached, it is considered that the mat has been rolled sufficiently because it is felt that over-rolling will cause checking or hair cracks.

At the present time the specifications of contracts now in force provide that a 3″ mat is to be placed, the same to be placed in two courses, but experience gained appears to indicate that just as satisfactory a surface can be obtained by placing the same in one course 2 or 2½″ thick. The original intent of the two course construction was to place the lower course as a leveling course to eliminate minor irregularities in the existing surface and then still be able to take out more irregularities in the upper course, thus providing a smooth riding final surface. The 1½″ depth was chosen because the maximum size of the aggregate was ¾″, thus providing a sufficient thickness for the maximum size of aggregate. However, in placing these so-called leveling courses, the same have been placed giving a good degree of smoothness and it is therefore felt that one course construction can be adopted of a slightly greater thickness and the final result accomplished at a lower cost. In connection with an extraction test, the department is attempting to develop a more rapid test for field use and so far the results obtained are encouraging. A brief description of this method follows:

AN APPARATUS FOR THE DETERMINATION OF PERCENTAGES OF BITUMEN IN ROAD MIXES

Heretofore, the available instruments for determining percentages of bitumen have been the Dulin Rotarex and the hot extractors of the New York type. Neither of these instruments is suitable when electric power is not available. Furthermore the first is too heavy for transportation in the field and the second requires lengthy operations before results are available; the time required is too long to be suitable for plant inspection work.

In order to make available an instrument with which quick determinations could be made consideration has been given to instruments which are based on the principle that light passing through a colored solution loses intensity in some proportion to the strength of the solution. None of the instruments available, however, are readily suitable to the measurement of bitumen-solvent solutions; besides, commercial instruments are

too expensive and delicate for field use. However, in order to make use of the principle in connection with bitumen-solvent solutions, an instrument consisting of the following has been devised.

1. A source of light of known intensity consisting of a flash light bulb (Mazda P. R.2, 0.5 Amperes, 2.4 Volts) mounted in a parabolic reflector. The bulb is energized by means of six No. 6 dry cells connected in series parallel to give 4.5 volts.

2. A photronic cell which receives the beam of light after it passes through the solution that is being measured.

3. A galvanometer of a sensitivity of 0.15 microamperes per millimeter.

4. Fixed resistors of 1000 and 1500 ohms which are in circuit with the photronic cell and the galvanometer for the purpose of controlling the sensitivity of the circuit and of "damping" the galvanometer.

5. A rheostat and a toggle switch which opens and closes the circuits.

6. A standard red filter which is used to protect the Photronic cell and to check the set up of the instrument.

7. Transmission cells which carry the solutions that are to be measured.

The apparatus, except the battery and galvanometer, is enclosed in a covered box containing one compartment for the light circuit and another for the Photronic cell and the resistors.

To get the instrument ready for use in the measurement of bitumen-solvent solutions, the following procedure is followed:

1. All connections are made tight and the wires connecting the various parts of the instrument are inspected to ascertain that there are no "shorts." The bulb is inspected to ascertain that it is in working condition.

2. The galvanometer needle is adjusted to read 0.0.

3. The standard filter is placed between the source of light and the Photronic cell. The current switch is thrown on and the amount of light passing through the filter is regulated by means of the rheostat until the galvanometer reads approximately 50.0. If this cannot be done the bulb and the dry cells are examined and replaced if necessary.

After setting the needle at 50.0, the bulb is allowed to remain lighted for at least five minutes but not for more than 10. By readjusting the rheostat and with the filter in position, the needle is made to read between 49.0 and 51.0. The exact reading is taken and identified as RF.

To take a reading with a bitumen-solvent solution the filter is moved forward and a transmission cell containing the solution is placed in position behind the filter. The latter is then removed, the box containing the instrument is closed, and a reading identified as RM is taken on the galvanometer.

The filter is then placed in position back of the transmission cell, the

latter is removed, the box is covered, and a check of the value of RF is then made—the reading should not vary by more than 0.1 millimeter.

The reading R is then calculated by means of the formula R=(RM-RF) plus or minus cell correction.

With the value for R, the percentage of bitumen is then read from the calibration tables furnished with the instrument.

The following illustrates readings and method of calculation.

RF = 49.7
RM = 51.6
Cell correction = −0.3
R = (51.6 - 49.7) −0.3 = +1.6

The value of 1.6 in the calibration table corresponded to a percentage of bitumen of 3.9.

The calibration tables are prepared by extracting the bitumen from mixes containing known percentages of bitumen. The following is a typical table.

R	% Bitumen
6.0	3.5
4.9	3.6
3.8	3.7
2.7	3.8
1.6	3.9
0.5	4.0

In order to obtain solvent-bitumen solutions suitable for use with the instrument just described, extractions are made as follows:

Weigh 500 grams of the bituminous mix into a 2 liter round bottom flask. Add 150 cubic centimeters of benzol, swirl for a second or two and allow to stand. At the end of 5 minutes swirl for one minute and decant into a 250 cc. graduate as much of the extract as possible. Note volume decanted and pour the extract into a large bottle. Using another 150 cc. portion of benzol repeat the extraction but allow 2 minutes instead of 5 for the period of standing, decant into the 250 cc. graduate, note the volume and pour extract into the large bottle. Repeat until 10 extractions are made. Add the volumes decanted and make up to 1500 cc. by the addition of benzol. Shake the solution vigorously and fill a pair of centrifuge tubes. (The centrifuge used is of the ordinary Babcock type). The solution is centrifuged until practically all the suspended mineral matter has been thrown down. The length and speed of centrifuging can be readily determined by trial.

By means of a pipette, transfer about 7 cc. of the centrifuged extract into a transmission cell and take a reading immediately.

The method of extraction and the apparatus herein described have been developed for use with road mixes containing not over 4.0% bitumen. If mixes of higher percentages are to be checked, the value for the factor RF can be varied to suit the need, and the method of extraction can be modified to suit the percentage and nature of the bitumen in the mix.

Stabilized Gravel Base and Bituminous Surfacing in North Dakota

By

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HISTORY OF STABILIZATION OF GRAVEL BASE ON FEDERAL AID PROJECT 217 AB RE

Using Blade Mix Method on First Course, and Traveling Plant Mix Method on Second Course

W. H. Noel Company started work on earth grading June 12, 1937, and finished October 14, 1937. The same contractor immediately started work on the stabilized base.

The first work done was that of knocking down the crown on the subgrade. This had a 6 inch crown on a 36 foot top. The stabilized mat has a 2½ inch crown on a 27 foot top with a 6 inch sluff on each side. This meant knocking 2 inches off the crown. The dirt obtained was to be used for the dirt shoulders and was windrowed on the shoulder. It had to be pushed down the inslope, however, as the job was suspended over the winter months; on the road the dirt would have caused snow trouble.

While the subgrade was being prepared, gravel hauling was started in Miles 3 & 4 on October 18. Specifications covering the combined aggregates are as follows:

"Gravel to be of durable fragments, mixed with a soil binder, the combination of which will give a P. I. on the fraction passing a No. 40 sieve between 8 and 15. The sum of the material passing the No. 40 sieve and P. I. shall not exceeed 50. The liquid limit of the fraction passing the No. 40 sieve shall not exceed the value determined from the formula: LL = 1.6 P. I. + 14. The combined material must also pass the following sieve analysis:

Passing 1 inch square mesh sieve	100%
Passing ¾ inch square mesh sieve	85-100%
Passing ⅜ inch square mesh sieve	65-100%
Passing No. 4 sieve	55- 85%
Passing No. 10 sieve	40- 65%
Passing No. 40 sieve	25- 50%

The above specifications were complied with by using a mixture of 15% clay and 85% gravel, using gravel from a pit located in the NE¼ of Section 36-153-57 and a mixture of 22% clay and 78% gravel, using gravel from a pit located in the SW¼ of Section 11-152-56. No difficulty was experienced in obtaining materials that conformed to specifications. The Testing Laboratory did reduce the clay percentage a trifle on the ground that the P. I.'s were too high.

The combined aggregate was hauled on the road in two equal courses of application, each being approximately 1275 cubic yards per mile (1020 cubic yards of gravel, 255 cubic yards of clay). The first course was hauled and laid dry in the fall of 1937. Subgrade preparation was the main difficulty encountered. Hauling had to be stopped to permit the blades to keep ahead. In hauling, some of the subgrade was cut by the trucks. This was remedied by applying water to the subgrade and packing it with pneumatic rollers.

P. I. tests on the first course showed results between 6.5 and 10.0, an average of about 8.5.

The first course was laid dry because weather conditions did not permit stabilization. The work was started in the fall to get a gravel surface for the winter and spring traffic, at the same time protecting the grade. The contract read that the first course had to be laid to section and stabilized if weather conditions permitted. Work was suspended for the winter months on November 2, 1937.

Work was resumed on March 29, 1938, and the immediate problem was to set up the first course. There was considerable float which had to be pounded into the mat. This was accomplished by blading all the float to one side, wetting down the firm portion to soften it, blading the float back on, wetting some more, and then packing the mat with pneumatic rollers. The first course seemed now to have a good compaction; tests showed densities between 135-142 lb./square foot. Difficulty was experienced in getting a good compaction test because of the thin section. Crank case drainings were used in lieu of Ottawa sand in obtaining compactions.

The work of hauling the second course clay and gravel was started April 20, 1938. This increment was in the same amount as the first course and in the same proportions. No difficulty was experienced in hauling the materials or in getting them to meet the specifications. The gravel was hauled first and a portion of the windrow was knocked down to permit the placement of the clay in the gravel; this was done to prevent loss of the clay. The combined material was bladed back and forth to insure proper dispersion of the clay throughout the gravel, since the Barber-Greene traveling plant would not thoroughly mix the ingredients itself.

The aggregate was then windrowed to a triangular section to facilitate pick-up by the traveling plant. To control the amount of water in the mix, the gravel influx is changeable and the water injection constant. The flow of gravel is governed by a gate over the conveyor dumping into the pug mill. When the mix emerges from the pug mill it has the

appearance of dry concrete. Traveling plants are excellent for mixing stabilized aggregates because they disperse the water and do not require it in as large quantities as in the blade mix method where the water is applied by distributors.

The next step was that of laying the material to section. After this more water was added and the pneumatic packers began to compact it. A small amount of float remained after the pneumatic packing; this was pressed in to the mat with a steel roller.

A few spots were found to have less than the required crown, other places had more. They were shaping by adding water and shaping with pneumatic and steel rollers. Where the crown was light, the entire mat was soaked and the shoulders packed while still wet. This forced the material in and built up the crown. As the material dried, the center was packed and the net result was a well-compacted mat with the desired crown. Where the crown was too great, this process was reversed; packing was started in the center and gradually worked out as the mat dried.

P. I.'s for the second course varied getween 6 and 9.5, averaging about 8.5. Compaction for the whole mat was found to be 135-140 lbs./square foot.

In putting the dirt shoulders on, it was found that the road was a trifle wide when the original inslope line was followed. We attribute this to the fact that possibly the first course was cut in more than the 2 inch crown cut. Pederson and myself were not present at this phase of the work, so we can not say for sure that such was the case. At any rate this was remedied by pushing dirt down the inslope to obain a 33 foot top. This made a secondary inslope that is correct in every respect.

All that now remained to be done was application of a tack coat. The contract originally specified 0.5 gal./square yard. Our Central Office at Bismarck changed this to 0.3 gal./sq. yard before the work was started. At first we were worried about the tack coat penetrating the base. We were held up for some time waiting for the mat to dry for one half inch of its depth, required by specifications, before applying the tack. Some difficulty was encountered with the tack coat picking up where the fines in the mat were not firmly embedded. The few places where this occurred were repaired by spot shooting. The shoulders were also tacked to keep moisture from seeping into the stabilized mat. In our opinion this is an excellent idea. The tack coat and mat broke up in spots when the traffic became heavy. This was repaired by patching with oil mix material after the oil mix contractor had moved on the job. Traffic should have passed over the stabilized gravel during all of the construction.

This would have shown up the weak spots and given ample opportunity to fix them.

Project Summary

Length of project	12.619 Miles
Thickness of mat	4½ inches
Gravel used	28,842 Cubic Yards

Clay used	6,326 Cubic Yards
Water applied	527.4 M. Gallons
Tack-coat (SC-2 applied)	76,472 Gallons

The original contract amount for the stabilized gravel surfacing was $37,194.82. This was decreased during the course of the work by $2,590.41 making the final cost $34,604.41.

Summary of Opinion

A stabilized gravel job should be open for traffic during its entire construction. Traffic will discover and show up any weak spots in the mat which otherwise would not be found until after the tack coat has been applied and the road thrown open to traffic. I believe it is a mistake to tack the stabilized gravel too long before the oil mat is laid. In this particular case the tack coat surface had to stand too long a period of traffic before the bituminous mat was laid over it. This was due to weather conditions encountered while the oil mix was being put on. The tack coat surface will not withstand excessive abuse on the part of traffic, thereby increasing subgrade preparation costs when the oil construction begins.

GENERAL DISCUSSION—STABILIZED GRAVEL BASE CONSTRUCTION BLADE MIX METHODS

The need for substantial and economical surface courses is recognized in all of our present highway planning. Experimental attempts to adequately surface existing grades has for the most part been ineffective though they have been beneficial. This is true partially because of the increased demands of the traffic carried and partially because of the experimental nature of the construction.

More work has been done and information gathered on surfaces since the sponsoring by the United States Bureau of Public Roads of compacting and stabilizing of roadbeds and gravel surfaces than ever before. Many very good results have been obtained and much reliable information established. While this work still goes on it is now almost universally standardized. The problem has now become one of balancing expenditures against service over a period of years for every type of highway and traffic.

The stabilized gravel base has been in use long enough to have proven that it is advantageous either as a wearing course or as a base for later surfacing. It provides a reliable all weather surface and one on which the problem of losing surface material due to traffic is minimized. It does have a maintenance problem in that it can be successfully manipulated for repairing for only a short time after rains have softened the surface, and at other times very little work is necessary or can be done. This means that unless special equipment is used to moisten the surface that under ordinary conditions and usual maintenance forces a very limited section could be taken care of in the short time the surface is workable following rains.

Standards of terminology and materials are now so well established

that every attempted stabilizing project can be correlated with work previously done and results observed. From this careful study, testing and selecting of materials used has become an essential part of this work. Here again one must reach a balance of conditions involved.

Experience has shown that proper grading with the highest possible resulting density is the most desirable condition for materials used. At the same time, usually, substantial construction can be accomplished using locally available materials. These do not always approach the desired grading and it is only because of the pliability of the method of improvement that good results can be obtained with material at hand. The use of locally available material aids materially toward economy and this makes the next consideration—just how far one may go toward controlling materials before the cost outweighs the benefits.

To illustrate the above case consider the added special provisions to North Dakota 1938 work, calling for stabilizing material pulverized to a fixed fineness and the percentage of material combined passing a 40 screen plus the P. I. of that portion not to exceed 50. The object of the above provision is obvious and there is no question as to their benefits. The effect upon contract unit prices is as would be expected. From this one must note that consideration from viewpoint of cost and construction detail must be coincident for good engineering of material requirements for every improvement contemplated.

The construction procedure for the average stabilized road has become a routine thing. Probably the most common and economical is the blade mix method. Care being taken to prevent loss of soil binder and excessive segregation of rock and picking up organic silt from the base, a more uniform mixture is obtained than by any other method.

Several types of plant mix operations have been tried but since patrols are necessary in the final laying to section and water needed during this operation, it appears that there is an unneeded duplication of equipment. A travelling plant working at the same rate of speed required to thoroughly mix oil mix on the road does not uniformly mix moist stabilized material. This is because the moment water strikes the material in the pugmill the stabilizer material tends to "ball up" and is in the finished mix in this form. The result is a mix, parts of which have too much clay and other parts containing none at all. If travelling plants are to be used, better results could perhaps be obtained by mixing the materials dry and applying water during the laying operations of the blade. This method would allow dry materials to thoroughly mix through the full length of the pugmill instead of the water removing the stabilizer material from the mix in the pugmill before mixing can take place. The same can apply to central stationary plant mix.

In all but the stationary plant method, the proportions of material to be used having been arrived at by preliminary investigation, the material can be windrowed on the roadway either by volume or weight in these proportions and the resulting mixture checked for desired results and corrections made if desired or necessary. In this respect it is well to bear in mind that, if the course is to be used for a wearing course, the P. I. of the material passing the 40 screen should be higher than

if oil is to be applied at once as high clay content is detrimental to application of oil.

The usual construction requires that the full course be placed in a series of lifts, the depth of which depends on the final thickness desired. That is, the number of lifts for a desired final depth shall be such as will give a convenient quantity of material to manipulate and not make a thickness which is too deep to compact well. This serves another purpose in that it prevents accumulated spots of poor material due to variations at source.

Next in line uniform moisture distribution is essential to good results. A wet and spongy portion, if worked until dry, will be very well compacted but will have an open textured surface high in clay content and checked in such a way that it soon fails. A dry spot will break up at once for it has no compaction or binding quality to hold it in place. Excessive water used to finish the surface produces a mortar film high in clay content which is slippery and ruts easily after rains and which glosses too highly when dry.

An application of enough water to soften the surface upon which material is to be laid is absolutely necessary. If this is not done a bond between the two is not established and the resulting plane of separation soon causes the top to break up under loading. This should be done prior to applying water to material in the windrow. This is done by any form of distributor available. In applying water to the windrow about ¼ gallon per square yard, if area of a layer is about ½″ deep, works well. A guide to proper moisture is appearance of dark spots in materials covering each application. When this has been done the material is moved enough to mix the moisture uniformly and to lay to desired section. Excessive blading is to be avoided as it brings rock to the surface, causing undesirable float on the finished surface.

Utilizing moisture of rains when previously dry mixed material is in the windrow helps for economy and is to be encouraged unless working forces are needed in maintaining work already completed. When laying is done this way the windrow must be moved enough to remove excessively wet spots and open up the dry area below the windrow so proper application of water may be made on this surface. In several cases the lines of failure correspond to the line of the windrow showing that proper care of this detail was not observed.

No set rules can be made as to rolling procedure other than that it should be slow, straight and from the outside toward the center of the section. An intelligent operator for this work can observe his progress and do much toward a well finished surface. He should be available at all times so as to work with the patrolmen while laying is in progress and roll as necessary after rains. Both steel and pneumatic types should be available.

The mat can be more thoroughly compacted by carrying traffic over portions laid to section. This will show up any defects so that they may be repaired before they are covered or involve extensive repair work. There is no object in pampering a project for appearance only. The

time to abuse it is during construction; after that load limits have a purpose. It might be well to give a base at least six months usage before covering it with any surface thus allowing for curing and exposure of weaknesses.

The type of section used should be either trenched or with thickened edge. While no beam action is expected in this surface the weakest portion and point of failure is usually the edge so it should be strengthened as much as possible. Feathered edges serve no roadway purpose but to hold adjacent material. Much material is lost over the shoulders due to traffic with this type of section.

When bituminous surface is to follow the base course, provision should be made for the edge of full thickness of base to be well beyond the line of the edge of the bituminous mat.

The crown of roadway should be well established and should never be below the common standard of ¼ inch per foot as the combined action of water and fast traffic is one of the severest abuses put to a road and crowning up to the point where it is noticeable to the driver of vehicles should be allowed to prevent this condition.

If shoulders are to be stabilized it must be expected that they will be used and they should be made heavy enough to stand ordinary traffic and constructed accordingly. Special care must be taken at any point changing mat thickness from shoulder to mat for traffic is indiscriminate in its use of roadway surfaces.

Because of the newness of this work, prices have been high. If the method and usage is to continue it must be made less expensive. If it is not the items of clay stabilizer and water which add nothing to the road but the means of initial shaping and compacting will have to give way to some low grade bitumen which will not only serve the same purpose but add moisture seal as well.

STATIONARY PLANT MIX BITUMINOUS CONSTRUCTION METHODS USE OF SC-7

Assuming that we have a stabilized base for an oil mix surface, we find that it may present any number of different aspects when application of prime or tackcoat is started. In some cases this has been applied previously and has been used as a wearing surface. This work having been done with the base course. Ordinarily this is not as successful as it might be as the base has not had a chance to dry and harden.

In the case where the tackcoat has been previously applied and used for any length of time as a wearing surface, it will usually be defective. There are sometimes small checks and cracks extending into the base. These may be remedied by a light application of bitumen to seal the openings. If the original application was made the base surface was glossy or high in clay or cold or wet, there will be a film of asphalt on the surface and no penetration. This film tends to peel off and further application cannot remedy this as it will not penetrate the layer of asphalt. When this condition exists there are usually holes in the base material due to action of water and traffic. These, if not too deep or frequent,

can best be filled with premixed oil material. If this is too involved the only alternative is to waste the tackcoat, reshape the surface and apply a new coat.

If the base has not been previously treated with oil, it may have a very dense and hard surface in which case it is usually somewhat irregular as to section because it is difficult to maintain this type of section. This is true because it takes a great deal of water to soften the surface enough to shape it. It is ordinarily better in this case to correct the irregularities with oil mix than to disrupt a substantial base. If this is done provision for extra material should be made and the tackcoat applied to the base as found. Especially is this true if a thin surface mat is to be used; otherwise, it is impossible to secure a good finished product as the mat tends to follow every fault of the base it is laid upon.

With a dense surface as above an application of SC-2 oil can not be very heavy or it will pool. Very little penetration can be expected and extended time on pooled oil does not seem to help this condition. The maximum penetration can be expected and extended time on pooled oil does not seem to help this condition. The maximum penetration seems to be obtained with the initial application. If the tackcoat is to be used as a wearing surface an MC oil is advantageous in that any excess oil not absorbed will be dried, soon forming a layer of asphalt for the wearing surface similar to a sealcoat layer. While with an SC oil, the time required for this is so much longer that the oil is usually carried away by traffic before this film forms.

With softer and recently constructed bases careful inspection for signs of breaking should be made before tackcoat is applied. Quite often openings will develop from the weight of the distributor. If this condition exists it should be remedied by further compaction before oil is applied, or oil applied to and mixed with the loose material until it will be tied down securely. If this is not done, moisture eventually gets below the tackcoat causing soft spots high in moisture which finally extend from bottom to top of the roadway surface and to seal this moisture up with oil cannot and should not be done as it will always break down.

When the base is thoroughly compacted, an application of SC oil up to the point where pooling starts, usually much higher than for the denser mats and about ½ gallon per square yard gives good results. It is well to apply all that the base will absorb for it helps tie it together and seal off moisture. Usually the best temperature of application is near the top of the allowable range, for the oil is then at its most liquid state and more easily absorbed. This effort is useless if the surface to be coated is cool or damp for the loss of heat is so rapid that an impenetrable film of asphalt forms at once and a tendency for this to "peel" off later is the result. The specification of a minimum air and surface temperature of at least 50 degrees F. is lenient and for good results should be closely observed.

Applying oil over bases with moisture in the interior results in spongy spots from which traffic removes the tackcoat and with it a thin layer of moist base material. In such cases the moisture must

eventually get out and will cause failure if oil is previously applied so observance of such portions should be made before applying the oil.

Applying tackcoat to dusty surfaces will result in formation of oil globules surrounded by dust particles. These will be picked up and carried away by traffic leaving the original dusty surface open. Succeeding applications will continue to do the same until all dust is removed so any dusty surface may as well be cleaned before initial application.

All free tackcoat oil must be adequately blotted before oil mix is placed as the mix usually has sufficient oil and has an affinity for free oil which causes an oil spot in the mat and an unstable condition which will bleed excessive oil and have to be removed.

With the use of SC-7 bitumen for mix oil in a standard stationary plant, extra provisions for heating the oil were made in most cases. A steam jacketed pipe line or preheater to the weighing bucket was provided and an ample return circulating circuit installed to prevent any accumulated asphalt due to cooling in the operation from storage tank. The temperature at which the oil was added in the pugmill was held in the range of 225 to 250 degrees Fahrenheit giving very good mixing results. It flows well and is taken up by the aggregates very well at this range and no loss in the properties of the oil was noticed. Every precaution must be taken to keep the tank cars and storage free of impurities and moisture or foaming of the oil at these temperatures will become a problem.

Very uniform results have been obtained with a 20 foot dryer, 30 foot elevator and 3000 lbs. pugmill using a 2400 pound batch and mixing time of thirty seconds. For any appreciable amount of work the plant units should not be smaller than this, for while they are not essential to obtaining proper temperatures they do provide ample time for moisture to escape and temperatures and mixing to be more uniform.

Using a clean, dense, hard aggregate in proportions of about 30 per cent retained on a quarter inch screen and 70 per cent passing and with 5 to 6 per cent of filler, material passing 200 mesh screen gave us a very densely graded mix which tests show to be from 118 to 120 pounds per cubic foot. The screen analysis being based on actual dry batches weighed through the plant.

With the above mix 2.9 to 4.0 per cent of SC-7 bitumen was added. With less clean material or softer constituents corresponding increase in this would be needed. The mix was dropped from the pugmill at about 200 degrees Fahrenheit and gave a good appearance. Moisture escaping from the mix can often be observed when it condenses on a cold batch thermometer placed in the mix. When this is excessive and the temperature of the mix drops, a large number of white uncoated rock are seen in the mix which indicates that the temperatures should not be lower or plant units any smaller or sufficient opportunity for mixing and drying will not be available.

The finished mix with all conditions in the desired range was on an average dark grey in appearance with all rocks well coated and glossy black. This tends to brown out slightly after being windrowed. Clay in

the gravel or excessive filler and moisture in the aggregates dulls the general appearance and causes uncoated rocks.

Upon being laid the mix takes on an early gloss but is very soft and is easily torn up by traffic, brake action or other severe abuse. While the mat is not considered as having strength, surface density varying in degree from maximum at the top to lesser degree toward the bottom is apparently developed and excessive loading at an early age usually causes many ruts and in some cases slight checking in the denser top surface. This is more apt to happen where a very hard base and thin mat tend to cause a creeping action of the mat. There is a tendency toward general roughness and pitting due to the slow curing nature of the bitumen. This might also account for checking and cracking in aged material since the asphalt content is such as might result in brittleness at certain stages. This condition is indicated and results as described when batches are heated above 250° F. to 300° F. If the mix is produced in the normal ranges set up it will harden with age and many of the irregularities disappear.

Excessive filler, over 7%, while not causing a prominent change in appearance of the mix usually is to be avoided as it results in a poor surface with small and numerous surface checks. Likewise a deficiency of fines in the grading causes excessive pitting, a poor surface and loss of metal in the surface. Deficiency in oil gives somewhat the same result and also a brown color. Burned batches being a light grey, having a slacked appearance. I have not seen this mix with so much oil as to result in bleeding and believe it would take a good deal of oil to be detrimental. From this it seems much better to operate on the high side of amount of bitumen added than to have deficiencies due to operating variances show up.

SEAL COAT APPLICATION
RC-2 and RC-3

Allowing a period of three weeks or more for curing and settlement and the appearance of defects should produce a surface suitable to seal. For a smooth, even and close textured surface RC-2 bitumen serves well as a sealcoat. It is best heated to the upper limits of its allowable range or 150° F. when its liquid condition gives very good coverage. Proper distributor nozzles, pressure and control are necessary. With nozzles giving a fine spray without heavy edge streaks and application made at the rate of ¼ gallon per square yard a good paint coat which will hold a fair amount of sealcoat sand is obtained. It is not heavy enough to hold chips.

If the surface of the mat is pitted or checked, RC-3 bitumen gives a heavier and stronger layer of asphalt and is to be recommended for such cases. Difficulty with end laps is experienced with this oil, and shutoff should be made as soon as the nozzles start to flicker giving a distinct and even end. Jacketed circulating distributor bars help in the use of this oil for instant nozzle starting is needed and difficult to get with this oil. Temperature of application as for RC-2 must be in the upper range of about 175° F. for this oil and for the reasons given for RC-2 oil. Proper precaution of open flames must be observed at this temperature.

This oil tends to bleed more than RC-2 oil, so excessive oil is to be guarded against and laps made as neatly as possible.

With mat containing SC-7 oil there is a tendency for any oil to soften and gloss the surface. Lubricating oil drops from vehicles take on a high luster and this would indicate that almost any type of oil could be used to shape, maintain or seal such a mat. Because of the softening effect mentioned more care must be shown while sealing this type of mat than others for any vehicle allowed on the sealcoat will pick up not only the sealcoat but a small part of the mat as well. This is more true with RC-3 oil than RC-2 as the consistency and stickiness of the oil is also a factor causing this until such time as the oil has set or been blotted.

The value of sealcoat aggregates for more than blotting is now accepted and indicated by increasingly rigid grading requirements. The special provisions now in use requiring 100 per cent passing the quarter inch screen and at least 80 per cent retained on the number 10 and containing no organic matter or excessive clay works well with either RC-2 or RC-3 oil. More rigid requirements than these should not be made however for this material is difficult to produce from the average local pit and anything more rigid would result in unreasonable prices.

Sealcoat aggregate should be fairly dry when applied and should be placed almost immediately after the oil or before the oil has acquired a surface film. If this is done each particle will pick up enough oil to bond it to the surface and blot the oil in this way. If the aggregate is wet this coating cannot take place and uniform distribution is hard to get resulting in very little adhering, and traffic soon whips the surface to a very high black polish. Slow traffic should be put over the sanded portions as soon as possible and manipulation of sand over the surface until no wet oily spots are visible and any other means of permanently placing the sand on the surface should be encouraged. This adds whiteness and wearing quality to the surface of benefit to traffic at night and when surface conditions are slippery. Rock chips have been found very satisfactory in blotting RC-3 oil and producing a surface of strong high metal character. The grading of those used on a test section was 100 per cent passing ½ inch screen, 20 per cent to 40 per cent passing ¼ inch screen and zero to 10 per cent passing a number 8 screen. It is desirable to have as much fracture material as possible but we found this quite low from local pit production methods.

Like sealcoat sand these chips were applied immediately after an application of RC-3 oil in amount slightly over a quarter of a gallon per square yard. This gave an opportunity to set down in the oil while still liquid. As soon as placed these were rolled with a 5 ton pneumatic roller which tended to carry stones from the surface, but a steel roller not being available it was necessary to use it. Much better results would probably have been obtained with a pavement roller. The rolling as done placed them in a good position and set them firmly in place, however, although not imbedding them in mat material.

As traffic dislodged material over the surface it was broomed uniformly and rolled at intervals during a twenty-four hour period. In the

end sufficient material tied strongly to the surface to blot the oil and carry the surface load on the aggregate, thus protecting the sealcoat proper. The surface resulting should be much lighter than the usual oiled top is under artificial lights because of the amount of aggregates on the surface. If it were not for the difficulty in producing this type of material from local sources, it should be used. However, from the average local pit the cost of production would result in prices beyond reasonable expectation of benefits.

BITUMINOUS SURFACING ON FEDERAL AID PROJECT 217 AB RE.
Using Traveling Plant Type Mix

Megarry Brothers, of St. Cloud, Minnesota, were awarded the contract for bituminous surfacing. They were prepared to begin work as soon as W. H. Noel Company completed the stabilized gravel surfacing, and moved on to the project on June 3, 1938.

The hauling of gravel aggregate was started on June 4 and completed on June 16. Eleven hundred tons of combined aggregates per mile were hauled on the project. The combined aggregates consisted of 92% pit run gravel and 8% filler (200 mesh material). This combination was derived from a sieve analysis of both the gravel and the filler. There was little variance from these percentages to obtain an average of 7.5% of 200 mesh material on the entire project. It was deemed advisable to keep this latter prcentage fairly high as there was a loss of the fines through excessive blading of the windrow for drying purposes. Some dusting was also encountered as the material passed through the traveling plant.

The filler used contained an average of 70% of material passing 200 mesh. The percentage of the same material in the gravel was approximately 2%. From the laboratory analysis of the gravel used, it was found that the shale content was 5.2% and that the percentage of clay and colloids was 8.8%.

The material specifications which governed this project are:

"The fraction of pit run aggregate passing a ten mesh sieve shall not contain more than 15% of clay colloids. The natural soil filler shall not contain more than 20% clay and colloids and 40% of the filler shall pass a 200 mesh sieve. The analysis of the combined aggregate shall fall in the following range:

Passing ¾ inch square mesh sieve	100%
Passing ¼ inch square mesh sieve	50- 75%
Passing No. 10 sieve	35- 65%
Passing No. 200 sieve	5- 14%

No difficulty was encountered in obtaining the proper gradation. The pit consisted of veins of coarse and fine gravel and by working these veins of material, the +¼ inch material was kept at a 30 to 35% average.

Two samples of the combined aggregate per mile were obtained from the windrow and screened. The data obtained was used in making curve sheets. The percentage of 200 mesh material for the entire project was between 5 and 7.5%. The percentage of material retained on the ¼ inch sieve was between 25% and 33%.

Hopper bottom trucks and end dump trucks were used for hauling gravel. A spreader box which spread the gravel to an 8 foot width and a depth of approximately 6 inches was attached to each truck as it dumped. The purpose of this spreader box was to even up the windrow and also to leave a thin layer upon which the filler could be dumped and spread, using a spreader box of 6 feet in width. This arrangement was not very successful, as the filler trucks had difficulty in pulling over the gravel layer. Rain occurred before much of the filler was placed on the road. Consequently, the gravel, previously hauled, was windrowed and then spread in a thin layer, as required for the spreading of the filler. All material was weighed on tested scales.

The traveling plant used was a Barber-Greene. The percentage of oil in the mix was controlled by a gate opening which varied the aggregate feed. The rate of oil pumped into the pugmill was kept constant by means of a gear governed pump. The above mentioned gate opening controlled the rate by which the aggregate was fed into the pugmill.

Mixing with the traveling plant was started June 20 and completed July 25. Difficulty was encountered at the start, as no one seemed to be able to judge the amount of bitumen necessary to complete the mix. This was due to the fact that the bitumen used was a heavier asphalt (SC-7) than any which had previously been used in road mixing. There was a large percentage of uncoated particles which caused the windrow to look very brown, notwithstanding the fact that more bitumen was added than would have seemed necessary had a hot mix been used. 4.5% bitumen was used in the first 4½ miles mixed. This percentage was later cut down to 4%, as the mix obtained by this latter percentage appeared to be sufficient in asphaltic content.

A great deal of inclement weather was met with during the construction period. Had the windrow of aggregate been below the 2% of moisture content required, the entire project could have been mixed in seven days. The speed of the machine varied btween 7.5 and 9 feet per minute. At this rate, one mile required approximately 10 hours of mixing time. In order to dry the windrow to a moisture content of less than 2%, blading was necessary, causing a loss of material which was impossible to recover with the blade.

Thirty five calendar days elapsed from the time actual mixing began until the mixing was completed. This excess time loss was due to rain and the consequent waiting for the windrow to dry out. The use of a motor patrol to aid and hasten the drying of the aggregate represented a large loss to the contractor, since it had not been included in his contract price. For this reason future bids on this type of work will be raised to compensate for probable adverse weather conditions.

Brooming was necessary to clean the tack coat surface. This was accomplished by using two men with hand brooms who swept the loose material into the windrow ahead of the traveling plant. The two men easily kept up with the machine. The Barber-Greene could not pick the material up from the base and leave a clean surface. It was at first thought that by blading and rolling the mixed windrow across the base that these fines would be incorporated into the mix. Such was not the

case; instead the fines were spread in a very thin film across the entire base and thereby hindered the bond between the tack coat surface and the mixed material. These fines could not be broomed off as they filled the cracks and minute depressions in the surface. To remedy this condition and enhance the bonding surface, a fog of tack coat bitumen had to be shot previous to laying. It was found that where this fogging had not been applied a slight failure of the oil mat occurred, caused by a movement of the mat on the base surface.

The bitumen was heated at the train siding and was taken out of the tank cars at approximately 290 degrees F. The oil was hauled to the plant by means of uninsulated distributors of 1200 gallon capacity. A large heat loss resulted in getting the oil to the road and into the storage tank of the Barber-Greene, which had a capacity of 700 gal. For this reason, to keep the temperature of the oil entering the pugmill between 250 degrees and 275 degrees, the burners on the Barber-Greene had to be operating constantly. Better results in mixing seemed to be accomplished at the above temperatures than at lower temperatures of the oil. Hot weather also aided the coating of a larger percentage of the aggregate and gave better results for mixing than cooler weather. The difference in temperatures between the dry aggregate entering the plant and the mixed material issuing forth was about 20 degrees.

The moisture content of the material dropped about 0.5% in the mixing process so that difficulty was encountered in drying the mixed material to the less than 1% moisture content required for laying.

Due to the heavy asphaltic content of the bitumen, the mixed material could not be maintained while laying. Traffic ruts were almost impossible to remove after the initial packing had taken place. This apparent primary set was also partly the cause of a large amount of uncoated float which was thrown to the shoulder by traffic. This float, composed of uncoated and partially coated ¼ inch + particles, could not be packed into the mat.

The quantity of material used, 1100 tons of combined aggregates per mile, gave a mat thickness of 1½ inches and a top width of 22 feet with a 6 inch sluff on both sides. The crown required was 0.17 feet or 2 inches. A pea gravel shoulder was placed on each side of the mat for a width of 5 feet. Sufficient clay, 11%, was added for compaction of the gravel shoulders.

The seal coat bitumen to be used was RC-2 oil. This was placed on the mat in two seals, the first being 0.15 gal./square yard, the second 0.30 gal./square yard of mat surface. The seal coat sand specifications required that 100% of the material pass the ¼ inch square mesh screen and not less than 80% be retained on a 10 mesh screen. A great deal of difficulty was met with in trying to screen out the fines during the process of manufacture. Consequently, the seal sand was shipped to the project by rail and consisted of washed material. 186 tons of seal sand to the mile was applied in two increments, one of 60 tons to the mile on the first seal, and one of 126 tons to the mile with the second seal. The intention of the two seal coat applications was to strengthen the

weaknesses that may develop in the thin mat. The first application was designed as a penetration and sealing coat, the second was to give added thickness to stand up under traffic wear. The contract originally provided for a seal of 0.25 gal./square yard with 101 tons of sand to the mile.

Liquid Bituminous Mix Using a Traveling Plant

By

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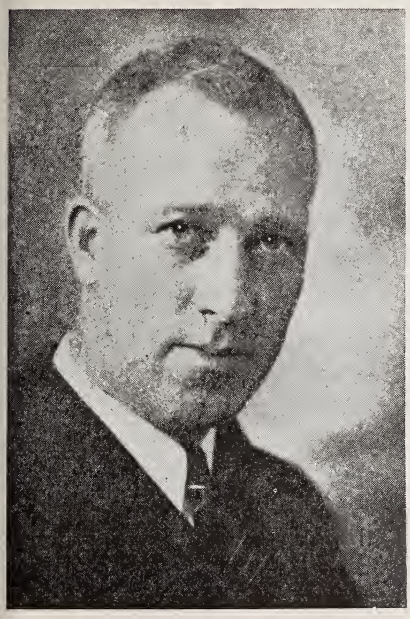

During this past season, the North Dakota State Highway Department has let to contract several liquid bituminous mix projects, permitting the successful bidder to use stationary plant, traveling plant, or blade mix methods. The liquid bituminous material specified was a very heavy grade and a good many contractors elected to do this work with various types of traveling plants. It is my intention to describe the type of materials used, methods of construction and contract costs on one particular project using a traveling plant.

This project is located in the northeastern part of the State of North Dakota on U. S. Highway No. 2 and is 12.619 miles in length and is known as Federal Aid Project 217 AB (Reopened) in Grand Forks County.

The grade was built during the 1937 construction season and one course of stabilized gravel completed. Early in 1938 the second course of stabilized gravel was completed. The gravel base was of the trench method type of construction four inches thick when compacted and twenty-seven feet wide on top with a six-inch taper on each side.

The contract for the stabilized gravel included a prime coat of SC-2 which was applied this last spring at the rate of three tenths (.3) gallon per square yard for a width of twenty-eight feet.

After the prime coat of SC-2 had completely penetrated gravel was hauled and placed in a windrow at the rate of 1012 tons per mile. This gravel was placed by a spreader box operated behind end dump trucks in order to obtain a uniform quantity of gravel per station.

A typical grading of the gravel material used is as follows:

Sieve	% Passing
¾	100
¼	64.7
No. 10	33.5
No. 28	18.1

No. 48	7.1
No. 80	3.6
No. 100	2.3
No. 200	1.3

The filler or 200-mesh material was added to this gravel, at the rate of 88 tons per mile, by means of a spreader box.

A typical grading of the filler used is as follows:

Sieve	% Passing
¼	100
No. 10	99.5
No. 28	98.8
No. 48	97.1
No. 80	93.5
No. 100	90.3
No. 200	78.6

After the filler was added a motor patrol placed the materials in a triangular shaped windrow in order that it might all be picked up by the traveling plant.

This windrow of gravel and filler was manipulated, when necessary, by a duck foot cultivator and blades until the surface moisture was less than two per cent by weight and again placed in a triangular shaped windrow.

The specifications of the mixture of gravel and filler are as follows:

Passing a ¾″ square mesh sieve	100%
Passing a ¼″ square mesh sieve	50- 75%
Pasing a No. 10 square mesh sieve	35- 65%
Passing a No. 200 square mesh sieve	5- 14%

After the moisture content was below the two per cent as required by the specifications, it was ready for the traveling plant and the addition of the liquid bituminous material.

The specifications of the liquid bituminous materials used for the prime and mix are as follows:

	Prime SC-2	Mix SC-7
Water	0.5–	0.5–
Flash, Cleveland Open Cup	200+	250+
Viscosity, Saybolt Furol @ 122° F.	100-200	
Viscosity, Saybolt Furol @ 140° F.		600- 1000
Loss on heating 50 grs., 5 hrs., @ 325° F.	9–	4–
Residue of 100 penetration @ 77° F.	60+	75+
Ductility of 100 penetration residue	100+	100+
Solubility in CCl_4	99+	99.5+

The traveling plant used on this particular project was a Barber-Greene built for use with heavy liquid bituminous materials. This was the first time a plant of this kind had been used to mix such a heavy liquid bituminous material in this State or any of the surrounding States

and consequently, there was considerable discussion pro and con whether or not satisfactory results could be obtained.

The bituminous material was shipped in insulated tank cars and arrived at its destination at temperatures varying from 90° F. to 160° F. Upon arrival on the project additional heat was applied to the SC-7 until it reached a temperature between 275° F. and 325° F. after which it was pumped into transport tanks and hauled to the traveling plant on the road. This traveling plant had storage tanks into which the SC-7 was transferred and additional heat applied when necessary.

It was found to be advisable to keep the temperature of the SC-7 between 275° F. and 300° F. at the time it was sprayed on the aggregates in the pug mill of the traveling plant. The aggregates should be kept in the pug mill as long as possible in order to obtain a satisfactory mix with SC-7.

As mentioned before, the specifications required less than two per cent surface moisture before the liquid bituminous material was added. On this particular project the aggregate temperatures ran as high as 108° F. and contained 1.7% moisture, after passing through the pug mill and being processed with SC-7 at a temperature of 290° F. the completed mix had a temperature of approximately 118° F. and contained 1.3% moisture.

On this particular project, the air temperatures at night were low enough to make mixing with SC-7 rather difficult and sometimes it was impossible to continue working all night.

After the mixed material had passed through the traveling plant, it was deposited in a uniform windrow behind the plant.

The base was treated with a tack coat of 1/10 to 1/12 gallon of SC-2 per square yard and the windrow was then moved onto this portion and the balance of the base given the same amount of tack coat.

After this was completed the windrow of mixed material was then ready to lay to the section specified on the plans. The inspector sampled the windrow, determined the moisture content and took samples of the mixed material to send to the laboratory for extraction. A typical extraction report from the laboratory is as follows:

Sample No.	14
Producer of Oil	Yale Oil Corp.
Date Received	7/15/38
Sampled from section	Sta. 340 — 365
Project No.	FAP 217 AB Reop.
County	Grand Forks
Lab. No.	OMX 1765
% of oil extraction	4.2
% passing ¾ sieve	100.0
% retained on ¼ sieve	28.5
% passing 10 mesh sieve	50.1
% passing 200 mesh sieve	6.7

The roadway was then staked to the width specified on the plans

and the mixed material laid to section with motor patrols. In laying the mixed material, the less manipulation required to bring the material to section the smaller the amount of segregation.

The bituminous mat was twenty-two feet wide at the top with a six-inch taper on each side, making the bottom width twenty-three feet, and had a thickness of one and one-half inches when compacted.

Sufficient mixed material was provided to widen all two to five degree curves one foot on the inside and to thicken the mat on the inside to two and one-half inches. This additional material required for widening and thickening is tapered back to the regular section at the P. C. and P. T. of the curves.

After the mixed material was laid to the section as specified the surface was rolled with a steel type roller and opened for traffic.

The shoulders of the roadway were then built up with gravel which was compacted. The quantity of gravel required to build up these shoulders was 326 cubic yards per mile.

After the traffic had been permitted to use this road from two to three weeks and any weak spots had been repaired, the seal coat was applied. The liquid bituminous material used for this seal coat was RC-2 applied at the rate of ¼ gallon per square yard twenty-three feet wide. One side of the roadway was treated at a time in order not to inconvenience traffic. A coarse, dry sand was then applied at the rate of approximately fifteen pounds per square yard by means of mechanical spreaders operated from trucks which moved backwards. The surface was then broomed with a drag broom in order to keep the coarse sand covering uniform. The dray type broom was used until the seal coat had taken all of the sand that it would hold.

The specifications for the sand and RC-2 for seal coat are as follows:

Sand for seal coat:

100% passing ¼″ sieve.
80% retained in a No. 10 sieve.

RC-2 for seal coat:

Flash point Tag open cup °F.	80+
Viscosity, Saybolt Furol @ 122° F.	200-400

Distillation, % by volume

Total distillate to—

437° F	10+
500° F.	15+
600° F.	20+
680–° F.	35+

Tests on residue from distillation—

Penetration @ 77° F., 100 G., 5 sec.	70-110
Ductility @ 77° F.	100+
Per cent soluble in CCl_4	99.5

The completed road had a twenty-two foot bituminous mat and a thirty-two foot finished top. The crown was seventeen hundredths (.17) feet on the bituminous section and an overall crown of twenty-four hundredths (.24) feet to the edge of the gravel shoulder. The inslopes on the road were 3½ to 1.

The contract prices for the project are as follows:

Furnish and apply SC-2 for base preparation	33,386 gals.	@	.075
Aggregates complete in place for bituminous mix	14,209 tons	@	1.35
Furnish and apply SC-7 for mix	176,192 gals.	@	.08
Rolling	75.714 mi.	@	2.50
Furnish and apply RC-2 for seal coat	42,672 gals.	@	.095
Sand for seal coat in place	1,279 tons	@	1.50
Gravel shoulders complete in place	4,194 cu yd.	@	.75

The mat produced by the heavy liquid bituminous material is very tough and appears that it will give very satisfactory service. However, it is thought advisable on future projects to require an RC-3 for seal coat and a sand which will all pass a ½ inch sieve and be retained on a No. 8, this sand to be applied at the rate of approximately 20 pounds per square yard. Then use a dray broom to uniformly distribute the sand and follow with a smooth steel roller.

Trends in Construction of Runways and Other Airport Surfaces

By

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INTRODUCTION

The surfacing of runways, taxiways, and hangar aprons and floors is a major phase in the development of airports. Surfacing is not, however, the only important phase in the development of land facilities for servicing and making practical use of the fast expanding airplane service, whereby this service now extends to practically all of the country's cities. Plans have been developed and will doubtless shortly be put into operation whereby shuttle planes will criss-cross the sky to extend air service even to tiny towns and villages. Therefore, airport construction in this country is far from completed.

"FLIGHT STRIPS"

In addition to need for constructing many more air fields and greatly enlarging the facilities at existing airports, there is a movement under way which is designed to actively develop more or less emergency landing fields, adjacent to the main highways. The Northwest Aviation Planning Council recently and formally adopted a resolution asking proper authorities in the Pacific Northwest to acquire sufficient land adjacent to the main highways to establish highway "Flight Strips" at intervals of fifty miles, each strip to be 300 feet wide and 3000 feet in length.

The Massachusetts House of Representatives have also considered "Flight Strips" adjacent to highways. That state's House Document No. 1540 contained the following extracts with respect to "Flight Strips":

"A 'flight strip' is a flat area, 200 to 800 feet wide, more or less, and 1,800 to 5,000 feet long, more or less, located adjacent to the highway, on publicly owned land. These 'flight strips' may be from 15 to 50 miles apart. The term 'flight strip' should not be confused with the statutory definition of the landing strip (emergency field), inasmuch as the 'flight strip' is included within the highway right of way, and is laid out and constructed by the same authority as is in charge

Note: All figures referred to are illustrated in Appendix of this Section.

of the highway. A 'flight strip' is not only adjacent to a highway, but is a part of the highway right of way itself, which is not necessarily the case of a landing strip.***

"As 'flight strips' relate directly to road transportation, which automatically becomes the responsibility of the various state highway departments, any state laws relative to 'flight strips' should be conceived as part of existing highway and motor vehicle laws.***"

Since most of you who are present are primarily interested in highway activities, it will be seen that there are at least some efforts being made whereby your work may be broadened to require your giving direct attention to planning, designing and supervising air fields of the "Flight Strip" type.

ONLY SURFACING CONSIDERED

Important as is the surfacing of runways, airport development includes other activities such as grading, drainage—of utmost importance—, lighting for illumination and signaling, also hangars and other buildings. Each of these phases requires expert planning and supervision if airport development is carried out successfully. For this paper, however, comments will be made only as to a description of trends and construction methods in the surfacing of runways, taxiways and other areas of a modern airport.

SURVEY EMPLOYED TO REVEAL TRENDS

Let me at the outset make clear that I shall confine my remarks concerning phases of design and construction of airport surfaces to facts which have been revealed to me by personal observations and replies to a questionnaire received from a number of municipal, military, naval and other authorities who have participated in the design, construction or operation of important air fields throughout the country. Also at the outset, I wish to express my appreciation and thanks to those engineers who so generously supplied me with comprehensive data and descriptive drawings.

LIGHTER, LESS EXPENSIVE SURFACES BEING USED

By far the most pronounced trend revealed in the replies and drawings received was that toward lighter and less expensive surfaces. Almost unanimously, engineers agreed that the heretofore accepted idea that heavy bases and wearing surfaces at airports were required was fallacious and unnecessarily expensive. This pronounced trend away from heavy expensive runway surfaces is best shown by the practice carried out at Port Columbus in Ohio (Fig. 1) where airport runway surfacing was pioneered more than ten years ago.

Figure 1 reveals that two programs of airport paving have taken place in Columbus. The design and cost of the first asphalt wearing course on Portland cement concrete base constructed ten years ago has now given way to a more rational and economical design in new runway extensions. This is illustrated by the fact that the most recent con-

struction called for 4½″ rolled limestone or slag base course instead of the more expensive 5″ Portland cement concrete base employed in the earlier program when matters of runway design had not developed to a point whereby actual impact and static loads could be accurately computed.

In this reversal of practice, the designers of Columbus' newest runway pavements more nearly approached the ideal economy sought in such matters, whereby the bearing ability of the layers separating the subgrade and the point of an airplane's contact should be made increasingly stronger from the allowable load on the subgrade to the point of the airplane's contact on the surface. This theory of design was recently commented upon in a paper in which the author*—in discussing highways—pointed out that no sharp planes of discontinuity should exist between these layers of increasing supporting value. The same principle applies perhaps even more strongly in considering runway paving than in the design of highways, for the reason that a runway's maximum load is static in nature because an airplane in landing creates little, if any, impact. This is due to the sustaining power of the airplane's wings, plus the cushioning effect of large tires and recoil types of springs in landing gears.

WEST COAST REVEALS SIMILAR TREND

Another striking illustration of this trend is revealed by a drawing (Fig. 2) sent by engineers at Reeves Field Airport (under the direction of the Harbor Department of the City of Los Angeles). This drawing, as shown in the cross-sectional view, reveals that while no change has been made in total thickness of runways, there has been a marked change whereby the cost of present-day construction at Reeves Field has been greatly reduced. This has been made possible by using available local materials and employing a type of asphalt construction less costly than that used in the earlier program.

As shown in the illustration (Fig. 2), a hot-laid asphaltic concrete base course and wearing surface having total thickness of 5″ was the design selected in the initial development of Reeves Field.

SUBGRADE-SURFACE THICKNESS RELATIONSHIP

Obviously, no fixed thickness of runway base and surfacing can be dogmatically prescribed for the reason that strength of subgrades vary so sharply in different localities due to varying soil and moisture conditions. Therefore, as in highway construction, the requirement is that the runway base and surfacing must be in proportion to the subgrade strength and the surface load which is expected.

Bernard E. Gray** had this requirement clearly in mind when he suggested the method of computing base-surface requirements on runways to be constructed on soils of varying bearing powers, as follows:

"For purposes of pavement design for determining the thickness

* C. S. Hill, Engineering News-Record, September 15, 1938.

** Chief Highway Engineer, The Asphalt Institute.

of structure, a value of 1.5 times static load may be used for W in the formula

$$t = .564 \times \sqrt{W/S} - e \quad \ldots\ldots\ldots \quad (1)$$

in which

$$W = \pi r^2 S$$

and from the large 45° triangle in the accompanying diagram

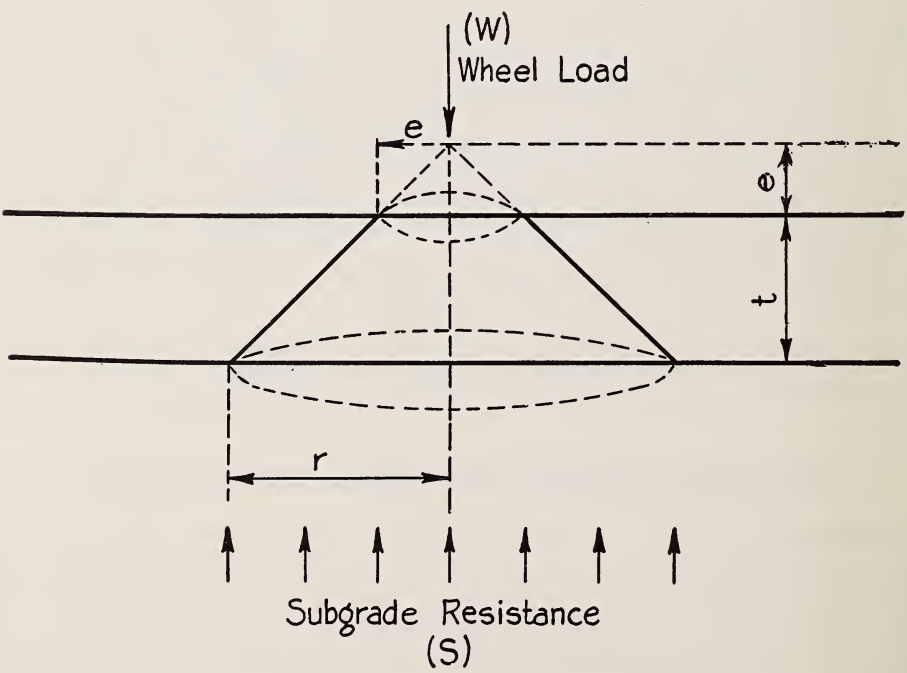
$$r = t + e$$

therefore

$$W = \pi (t+e)^2 S \quad \ldots\ldots\ldots\ldots (2)$$

Solving for t, the thickness of structure can be obtained from (1) where

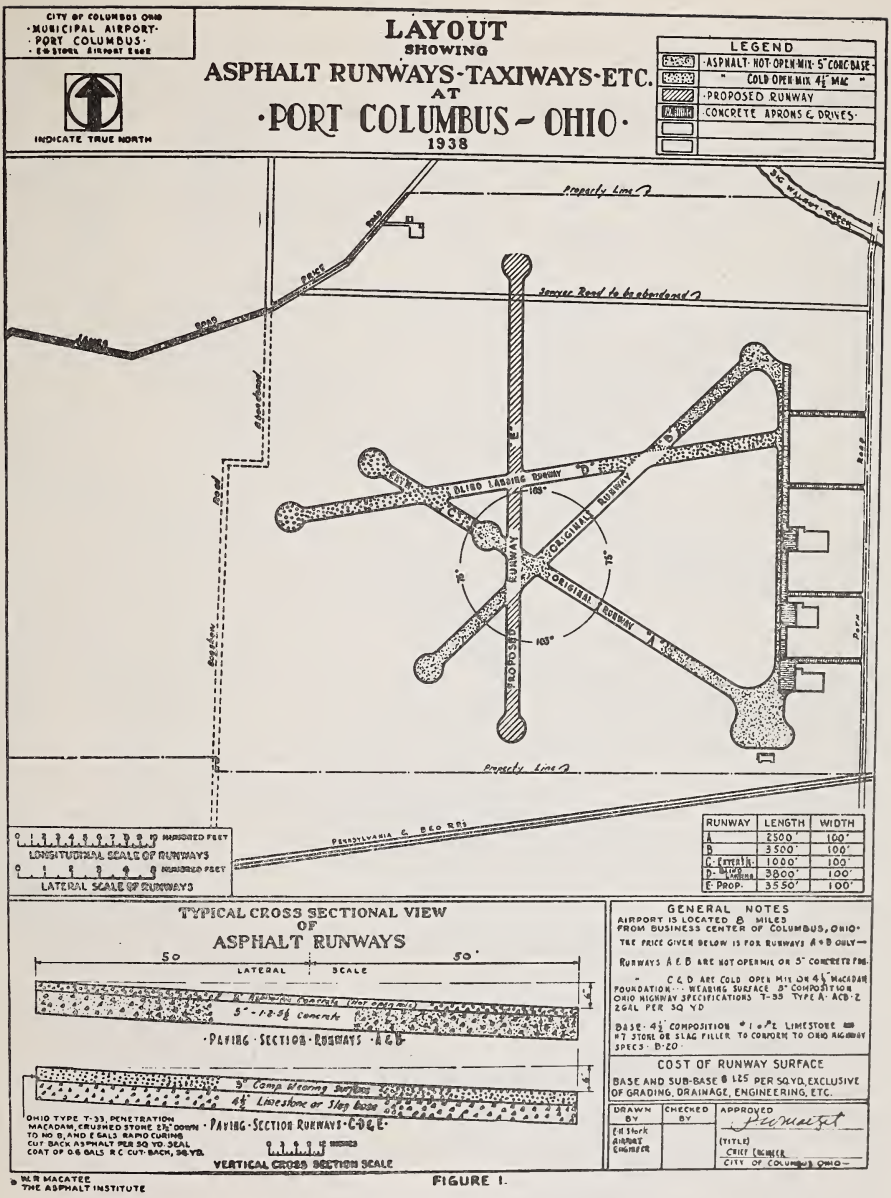
t = thickness of pavement (in).
W = wheel load (lbs.)
r = radius of circle of load distribution
πr^2 = area of circle of load distribution
S = subgrade resistance (lbs./in.^{2})
e = radius of tire area contact



Using a value of 1.5 times static load and with the other factors, an adequate margin of safety will be provided.''

AIRPORT SURFACES TO FIT COMMUNITIES' NEEDS

An interesting comparison of two airports' capacities—both in respect to number of planes incoming and outgoing in given periods and base-surface thicknesses—is made possible by studying the details of a small airport (Fig. 4) located at Chesapeake, Ohio in the southeastern-


CITY OF COLUMBUS OHIO
MUNICIPAL AIRPORT
PORT COLUMBUS
E&S STORE AIRPORT EDGE
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
PORT COLUMBUS - OHIO
1938
LEGEND
ASPHALT HOT OPEN MIX 5" CONC BASE
" COLD OPEN MIX 4½" MAC "
PROPOSED RUNWAY
CONCRETE APRONS & DRIVES
Property Line
Sewer Road to be abandoned
OHIO LANDING RUNWAY "D"
ORIGINAL RUNWAY "C"
ORIGINAL RUNWAY "A"
PROPOSED RUNWAY "E"
105°
75°
103°
Pennsylvania & B&O RRs
HUNDRED FEET
LONGITUDINAL SCALE OF RUNWAYS
LATERAL SCALE OF RUNWAYS
RUNWAY | LENGTH | WIDTH
A | 2500' | 100'
B | 3500' | 100'
C-Extnsn | 1000' | 100'
D-Ohio Landing | 3800' | 100'
E-Prop. | 3550' | 100'
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
50'
LATERAL SCALE
50'
5"-1:2:4 Concrete
PAVING SECTION RUNWAYS A & B
5" Comp Wearing Surface
4½" Limestone or Slag base
OHIO TYPE T-33 PENETRATION MACADAM, CRUSHED STONE 5½" DOWN TO NO 8, AND 6 GALS RAPID CURING CUT BACK ASPHALT PER SQ YD SEAL COAT OF 0.6 GALS R C CUT-BACK, PER YD.
PAVING SECTION RUNWAYS C D & E
INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 6 MILES FROM BUSINESS CENTER OF COLUMBUS, OHIO.
THE PRICE GIVEN BELOW IS FOR RUNWAYS A & B ONLY —
RUNWAYS A E B ARE NOT OPEN MIX OR 5" CONCRETE PVG.
— C & D ARE COLD OPEN MIX OR 4½" MACADAM FOUNDATION — WEARING SURFACE 3" COMPOSITION OHIO HIGHWAY SPECIFICATIONS T-33 TYPE A-ACB-2 2GAL PER SQ YD
BASE 4½" COMPOSITION #1 & #2 LIMESTONE OR #7 STONE OR SLAG FILLER TO CONFORM TO OHIO HIGHWAY SPECS B-20
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $ 1.25 PER SQ YD, EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
(TITLE)
CITY ENGINEER
CITY OF COLUMBUS, OHIO

W. R. MACATEE
THE ASPHALT INSTITUTE

FIGURE 1.

most corner of the state. A comparison with the much larger Chicago Municipal Airport (Fig. 3) would dwarf the Chesapeake project unless a study were made of the relative needs of the two communities which these airports serve. Such a study would likely show that the smaller project was designed with equal care and consideration for the purpose of adequately serving the requirements of a smaller population in the cities of Ironton in Ohio, Huntington across the river in West Virginia and Ashland, Kentucky.

REFUSE MATERIALS SERVE THREEFOLD PURPOSE

The materials which are shown to have been used in the sub-base of Chicago's big airport (Fig. 3) of more than one million square yards of surfacing, consist of waste materials removed from old street bases and surfacings which were badly broken up due to long previous use. These materials were of various character; old spall paving, broken-up concrete

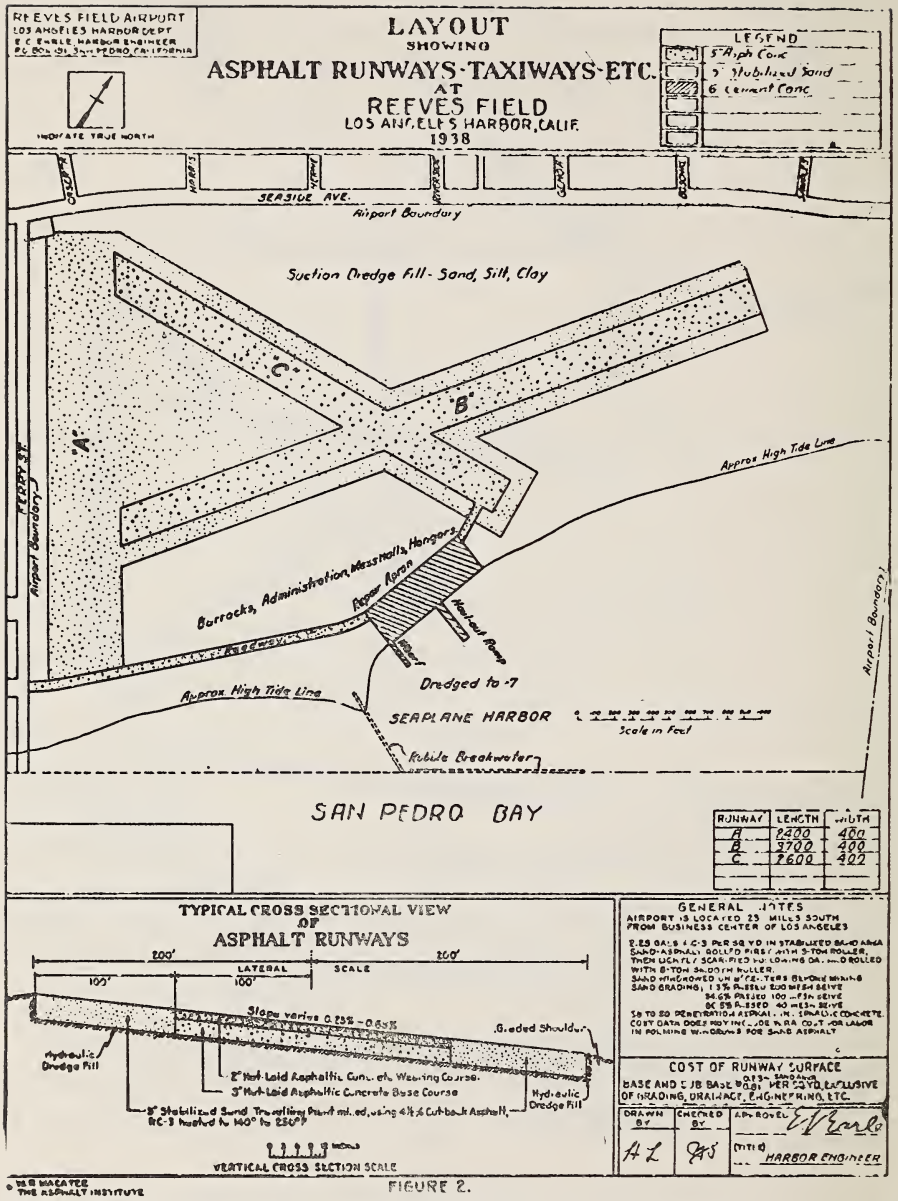


FIGURE 2.

and slabs of sheet asphalt which had been removed from many sections of the city.

These waste materials served admirably to stabilize the existing poor subgrade conditions at the city's airport. They further served to provide work-relief for unemployed persons engaged in street reconstruction. Of equal importance, the airport area served admirably as a depository for these materials which otherwise would of necessity have been hauled greater distances at added expense without, however, serving as useful a purpose.

Strictly speaking, there was probably no structural necessity for using as much materials in the sub-base as were used. Under the circumstances as outlined, however, they serve admirably. This is especially true due to the very unstable soils encountered in the section of Chicago where the airport is located.

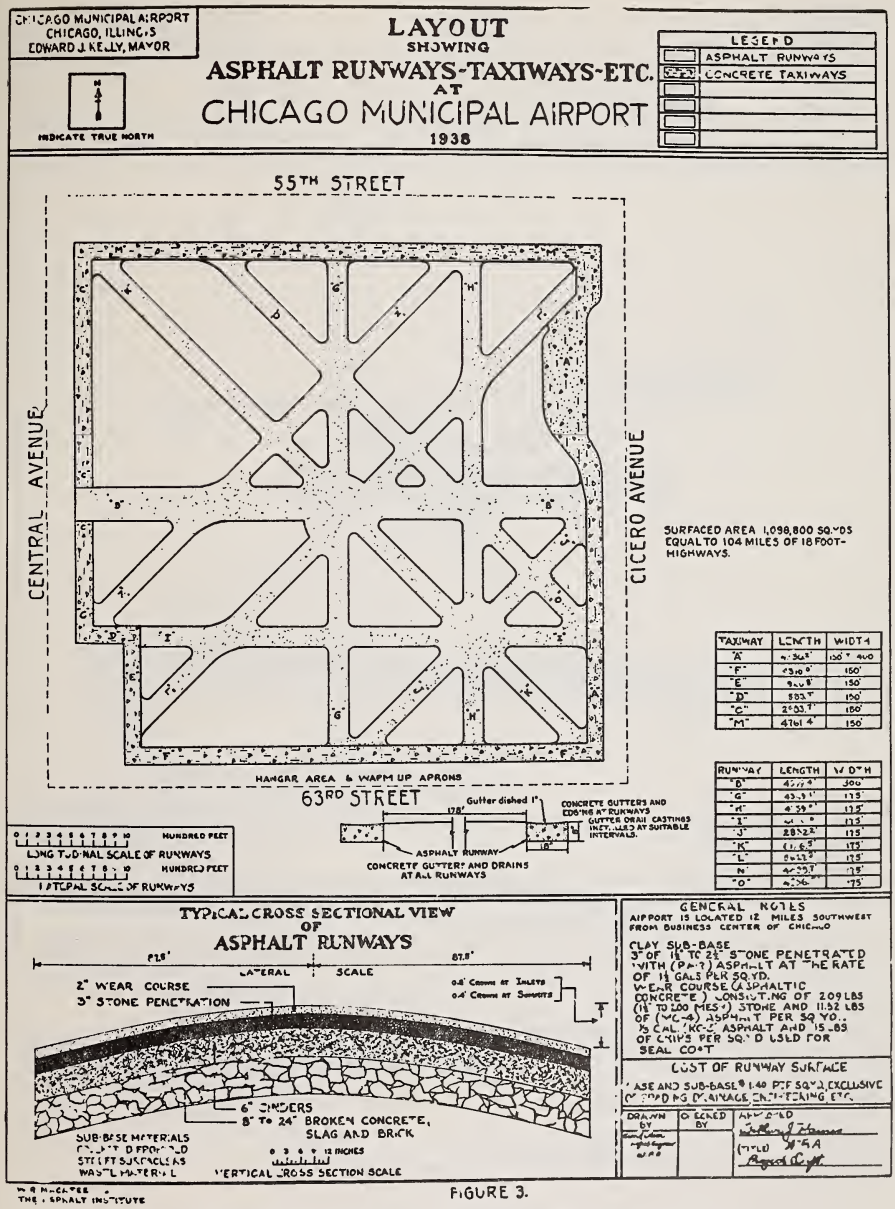

CHICAGO MUNICIPAL AIRPORT
CHICAGO, ILLINOIS
EDWARD J. KELLY, MAYOR
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
CHICAGO MUNICIPAL AIRPORT
1938
INDICATE TRUE NORTH
LEGEND
ASPHALT RUNWAYS
CONCRETE TAXIWAYS
55TH STREET
CENTRAL AVENUE
CICERO AVENUE
63RD STREET
HANGAR AREA & WARM UP APRONS
SURFACED AREA 1,098,800 SQ. YDS EQUAL TO 104 MILES OF 18 FOOT-HIGHWAYS.
TAXIWAY | LENGTH | WIDTH
RUNWAY | LENGTH | WIDTH
LONGITUDINAL SCALE OF RUNWAYS
LATERAL SCALE OF RUNWAYS
HUNDRED FEET
Gutter dished 1"
CONCRETE GUTTERS AND CURBING AT RUNWAYS
GUTTER DRAIN CASTINGS INSTALLED AT SUITABLE INTERVALS.
ASPHALT RUNWAY
CONCRETE GUTTERS AND DRAINS AT ALL RUNWAYS
TYPICAL CROSS SECTIONAL VIEW OF ASPHALT RUNWAYS
LATERAL SCALE
2" WEAR COURSE
3" STONE PENETRATION
6" CINDERS
8" TO 24" BROKEN CONCRETE, SLAG AND BRICK
SUB-BASE MATERIALS
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 12 MILES SOUTHWEST FROM BUSINESS CENTER OF CHICAGO
CLAY SUB-BASE
3" OF 1½" TO 2½" STONE PENETRATED WITH (PAR-?) ASPHALT AT THE RATE OF 1½ GALS PER SQ.YD.
WEAR COURSE (ASPHALTIC CONCRETE) CONSISTING OF 209 LBS (1½ TO 200 MES.) STONE AND 11.52 LBS OF (MC-4) ASPHALT PER SQ YD.. 15 CAL. KC-5 ASPHALT AND 15.83 OF CHIPS PER SQ. YD USED FOR SEAL COAT
COST OF RUNWAY SURFACE
BASE AND SUB-BASE @ 1.40 PER SQ. YD. EXCLUSIVE OF ROADWAYS, DRAINAGE, ENTRANCE, ETC.
DRAWN BY
CHECKED BY
APPROVED
(TITLE) M.S.A.
W. R. McCATEE
THE ASPHALT INSTITUTE

FIGURE 3.

ENRICHED UPPER SURFACE DESIRABLE

Engineers responsible for designing and constructing the big metropolitan Salt Lake City Airport (Fig. 5) and the Chanute, Kansas Municipal Airport (Fig. 6) both employed an excellent principle to follow in constructing asphalt runways. It will be observed that while types of surface construction were quite different—one using hot-mix type of asphalt, whereas the other used a liquid asphalt—each of the engineers apparently recognized the need for enriching the exposed upper surfaces of the runways, using somewhat greater quantities of asphalt than would normally be required on roadway surfaces.

It might appear as though the designers of the runways at Chanute, Kansas, subordinated surfacing to base stability and its integrity. However, keeping in mind the ill-effects of surface moisture, air and wind—

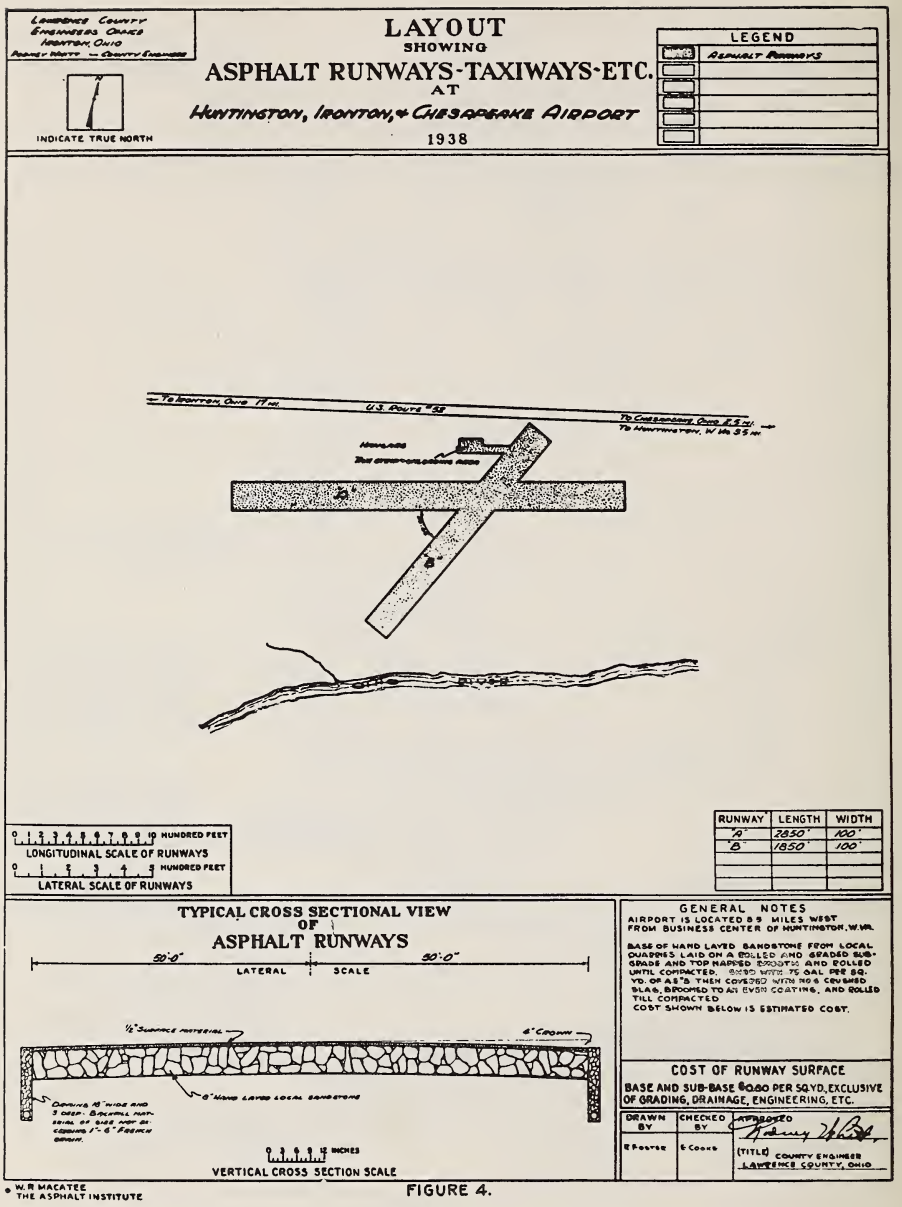

LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
HUNTINGTON, IRONTON, & CHESAPEAKE AIRPORT
1938
LEGEND
ASPHALT RUNWAYS
INDICATE TRUE NORTH
LONGITUDINAL SCALE OF RUNWAYS
LATERAL SCALE OF RUNWAYS
RUNWAY | LENGTH | WIDTH
'A' | 2850' | 100'
'B' | 1850' | 100'
TYPICAL CROSS SECTIONAL VIEW OF ASPHALT RUNWAYS
50'-0"
LATERAL SCALE
50'-0"
6" CROWN
6" HAND LAYED LOCAL SANDSTONE
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 6 5 MILES WEST FROM BUSINESS CENTER OF HUNTINGTON, W.VA.
BASE OF HAND LAYED SANDSTONE FROM LOCAL QUARRIES LAID ON A ROLLED AND GRADED SUB-GRADE AND TOP RAKED SMOOTH AND ROLLED UNTIL COMPACTED. SURFACE WITH .75 GAL PER SQ. YD. OF A8"S THEN COVERED WITH NO 6 CRUSHED SLAG, BROOMED TO AN EVEN COATING, AND ROLLED TILL COMPACTED
COST SHOWN BELOW IS ESTIMATED COST.
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $0.80 PER SQ.YD. EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
E Foster
E Cooke
(TITLE) COUNTY ENGINEER
LAWRENCE COUNTY, OHIO
W. R MACATEE
THE ASPHALT INSTITUTE

FIGURE 4.

the latter is especially damaging in areas like Kansas—it will be seen that the surface course was not unduly neglected, thin though it may appear. To prevent, so far as possible, the damaging effects of the elements, the designers of the Chanute runways used the relatively large quantity of 1.1 gallons of asphalt per square yard—including tack coat on macadam base—for the relatively thin wearing course, ½″ thick. This created an impermeable surface without, however, reducing stability below the minimum required to prevent displacement under the traffic to be expected.

SOFT ASPHALT DESIRABLE

It will be seen that the same principle was followed in constructing the upper wearing surface of the asphalt runways at Salt Lake City (Fig. 5). At that important transcontinental airport, the hot-mixed asphaltic concrete was thoroughly sealed with a rich mixture of sheet asphalt, using

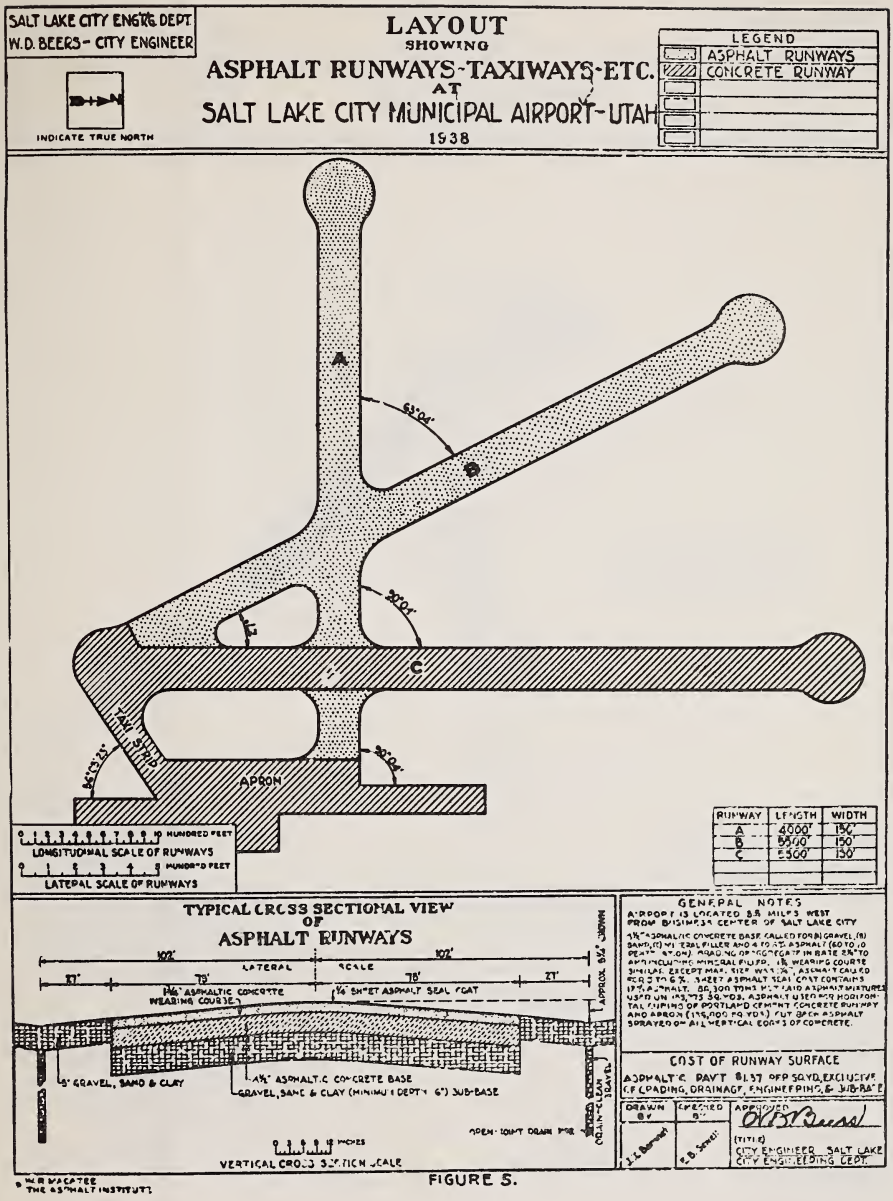


FIGURE 5.

12 per cent of relatively soft asphalt, 60/70 penetration. Not only is it desirable to use as high a percentage of asphalt as possible without unduly reducing the stability whereby displacement might occur, but it is equally desirable to use as soft an asphalt as possible consistent with minimum stability.

ASPHALT STABILIZATION EXTENSIVELY USED

From the replies received from a number of engineers throughout the country, it was made quite evident that asphalt stabilization of existing soils is being used more and more extensively, and with increasing confidence that it provides satisfactory and economical base courses for runways. In many cases asphalt stabilization has been carried out with no additions of hauled-in aggregates. In some cases, however—if the costs justified—asphalt stabilization has been carried out by means of a combina-

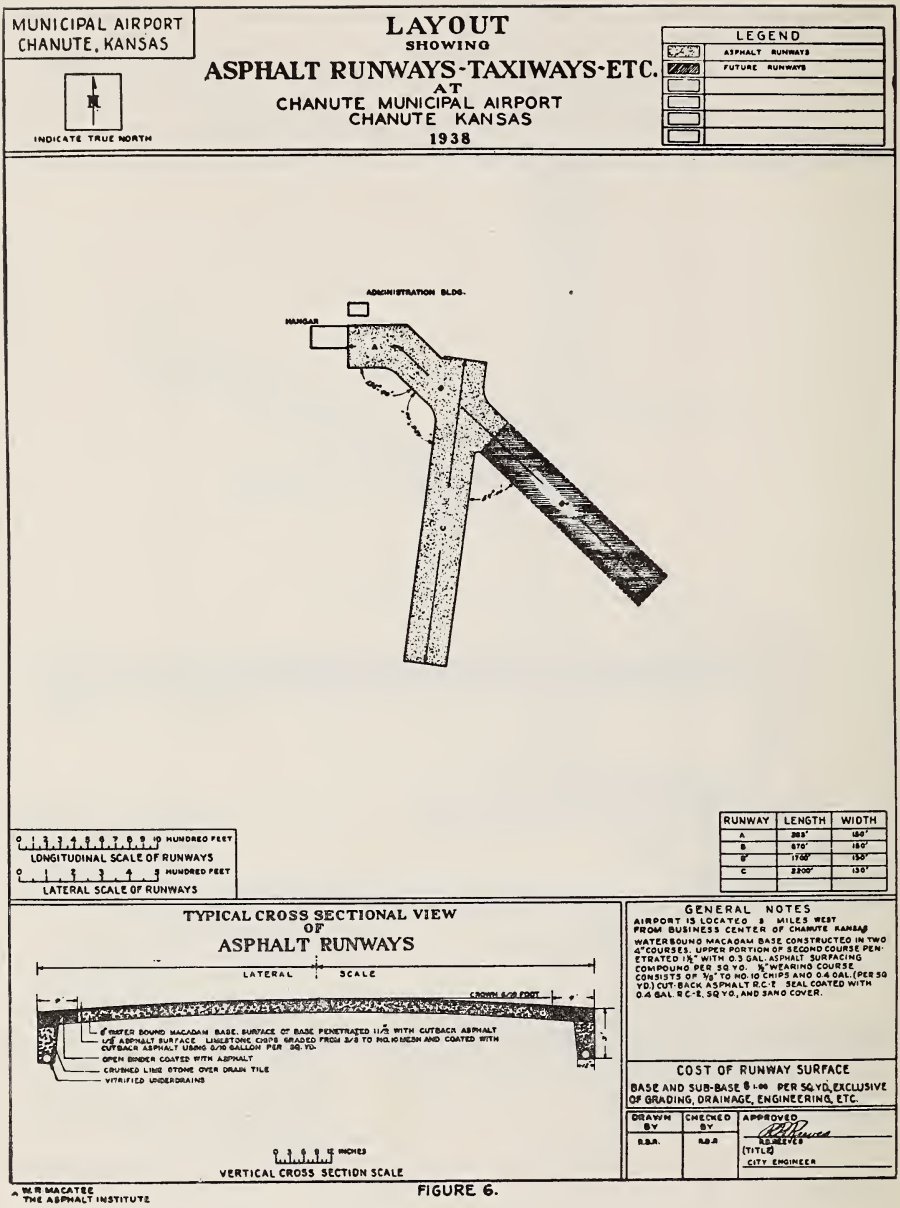
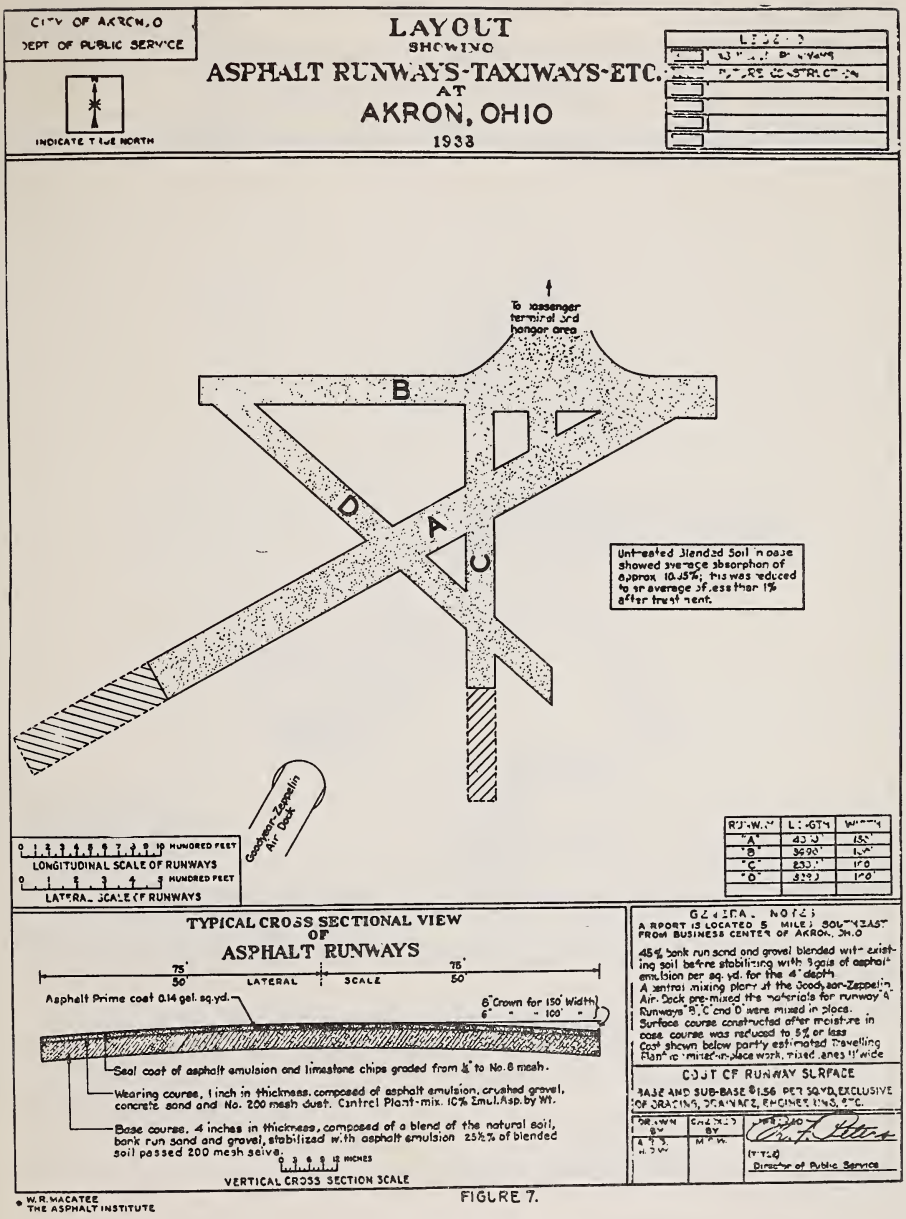

MUNICIPAL AIRPORT
CHANUTE, KANSAS
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
CHANUTE MUNICIPAL AIRPORT
CHANUTE KANSAS
1938
LEGEND
ASPHALT RUNWAYS
FUTURE RUNWAYS
ADMINISTRATION BLDG.
HANGAR
HUNDRED FEET
LONGITUDINAL SCALE OF RUNWAYS
HUNDRED FEET
LATERAL SCALE OF RUNWAYS
RUNWAY | LENGTH | WIDTH
A | 2855' | 150'
B | 2570' | 150'
B' | 1760' | 150'
C | 2200' | 150'
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
LATERAL SCALE
CROWN 5/16 FOOT
WATER BOUND MACADAM BASE, SURFACE OF BASE PENETRATED 1 1/2" WITH CUTBACK ASPHALT
1/2" ASPHALT SURFACE LIMESTONE CHIPS GRADED FROM 3/8 TO NO.10 MESH AND COATED WITH CUTBACK ASPHALT USING 5/10 GALLON PER SQ. YD.
OPEN BINDER COATED WITH ASPHALT
CRUSHED LIME STONE OVER DRAIN TILE
VITRIFIED UNDERDRAINS
INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 2 MILES WEST FROM BUSINESS CENTER OF CHANUTE KANSAS
WATER BOUND MACADAM BASE CONSTRUCTED IN TWO 4" COURSES, UPPER PORTION OF SECOND COURSE PENETRATED 1 1/2" WITH 0.3 GAL. ASPHALT SURFACING COMPOUND PER SQ YD. 1/2" WEARING COURSE CONSISTS OF 1/2" TO NO. 10 CHIPS AND 0.4 GAL. (PER SQ YD.) CUT-BACK ASPHALT R.C.-2 SEAL COATED WITH 0.4 GAL. R.C.-2, SQ YD, AND SAND COVER.
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $ 1.00 PER SQ YD, EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
R.S.A.
R.S.A.
A.B. REEVES
(TITLE)
CITY ENGINEER
W. R. MACATEE
THE ASPHALT INSTITUTE

FIGURE 6.

tion of (a) existing soils, (b) hauled-in aggregates and (c) asphalt. That is, by the incorporation of (a) local soils, (b) hauled-in, cheap aggregates and (c) asphalt, satisfactory bases have been produced by means of a combination of mechanical stabilization and structural stabilization.

This dual-form of stabilization is illustrated in drawing representing the Cheyenne, Wyoming Airport (Fig. 9-A). It will be observed in that instance that a gravel sub-base was installed prior to placing the stabilized base course in which cutback asphalt MC-3 was the stabilizing element. An outstanding airport project in which asphalt also was used for stabilizing blended soils is illustrated in the drawing for Akron, Ohio. (Fig. 7).

Another interesting and economical use of asphalt for stabilizing local soils was carried out at the Municipal Airport at Charleston, South Carolina (Fig. 8). The nomenclature employed in referring to work


CITY OF AKRON, O
DEPT OF PUBLIC SERVICE
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
AKRON, OHIO
1938
To passenger terminal and hangar area
B
D
A
C
Untreated Blended Soil Passage showed average absorption of approx. 10.45%; this was reduced to an average of less than 1% after treatment.
Goodyear-Zeppelin Air Dock
LONGITUDINAL SCALE OF RUNWAYS
LATERAL SCALE OF RUNWAYS
HUNDRED FEET
RUNWAY
LENGTH
WIDTH
"A" 4030' 150'
"B" 3690' 150'
"C" 2330' 100'
"D" 3390' 100'
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
75'
50'
LATERAL SCALE
Asphalt Prime coat 0.14 gal. sq.yd.
6" Crown for 150' Width
4" " 100' "
Seal coat of asphalt emulsion and limestone chips graded from ½" to No.8 mesh.
Wearing course, 1 inch in thickness, composed of asphalt emulsion, crushed gravel, concrete sand and No. 200 mesh dust. Control Plant-mix. 10% Emul. Asp. by Wt.
Base course, 4 inches in thickness, composed of a blend of the natural soil, bank run sand and gravel, stabilized with asphalt emulsion 25½% of blended soil passed 200 mesh sieve.
INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
A REPORT IS LOCATED 5 MILES SOUTHEAST FROM BUSINESS CENTER OF AKRON, OHIO
45% bank run sand and gravel blended with existing soil before stabilizing with 9 gals of asphalt emulsion per sq. yd. for the 4" depth.
A central mixing plant at the Goodyear-Zeppelin Air Dock pre-mixed the materials for runways "A" Runways "B", "C" and "D" were mixed in place.
Surface course constructed after moisture in base course was reduced to 5% or less
Cost shown below partly estimated Travelling Plant re-mixed-in-place work, mixed lanes 11' wide
COST OF RUNWAY SURFACE
BASE AND SUB-BASE @ 1.56 PER SQ.YD. EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
H. J. W.
M. P. W.
Director of Public Service

W. R. MACATEE
THE ASPHALT INSTITUTE

FIGURE 7.

similar to that which was done in Charleston, whereby it is designated as sand-asphalt by many persons and asphalt stabilization by others, indicates the need for clarifying these terms which are used so often interchangeably without always completely revealing their meaning. Kelley† comments on this as follows:

> ". . . a definite distinction is made between bituminous-stabilized soil bases and those types of construction in which soil materials composed principally of sand are used to produce what are essentially asphaltic paving mixtures."

While it is probably desirable, as Kelley implies, to more clearly define these terms which are used interchangeably, the work at Charleston

† E. F. Kelley, Chief, Division of Tests, U. S. Bureau of Public Roads.

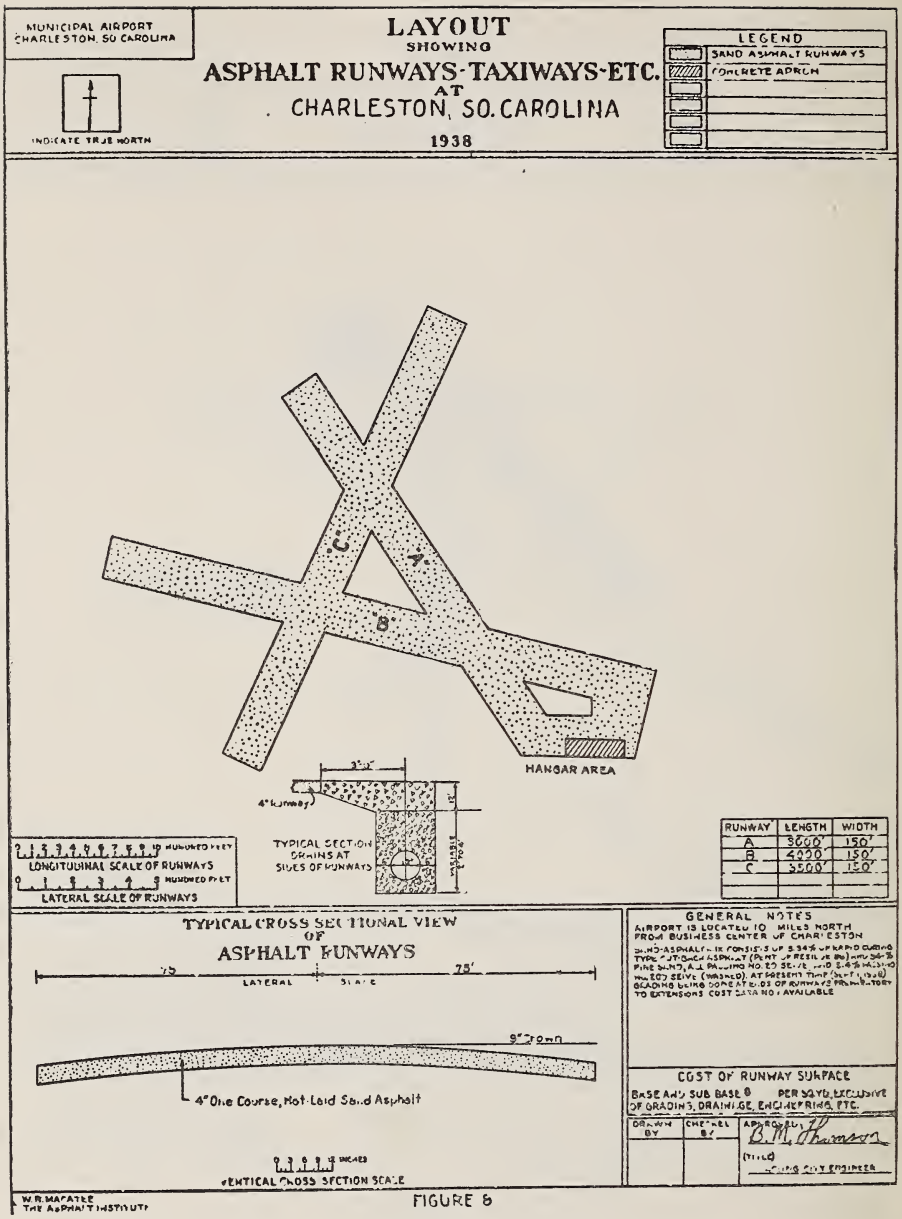


FIGURE 6

nevertheless does constitute a form of asphalt stabilization wherein the available local sands were made stable and satisfactory as runways after processing them with asphalt of the rapid-curing type.

MILITARY-NAVAL AIRPORTS USE ASPHALT STABILIZATION

The Army Air Corps Tactical School runways at Maxwell Field, Alabama, (Fig. 9) illustrates the use of emulsified asphalt for stabilizing an admixture of coarse sand which was added to the existing soil found at that army airport.

In contrast, the naval engineering authorities at the nearby Air Station at Pensacola, Florida used the slow-curing type (SC-3) for work at Corry Field to stabilize combination of clay and loam. Another air field serving the Pensacola Naval Air Station is identified as Chevalier Field.

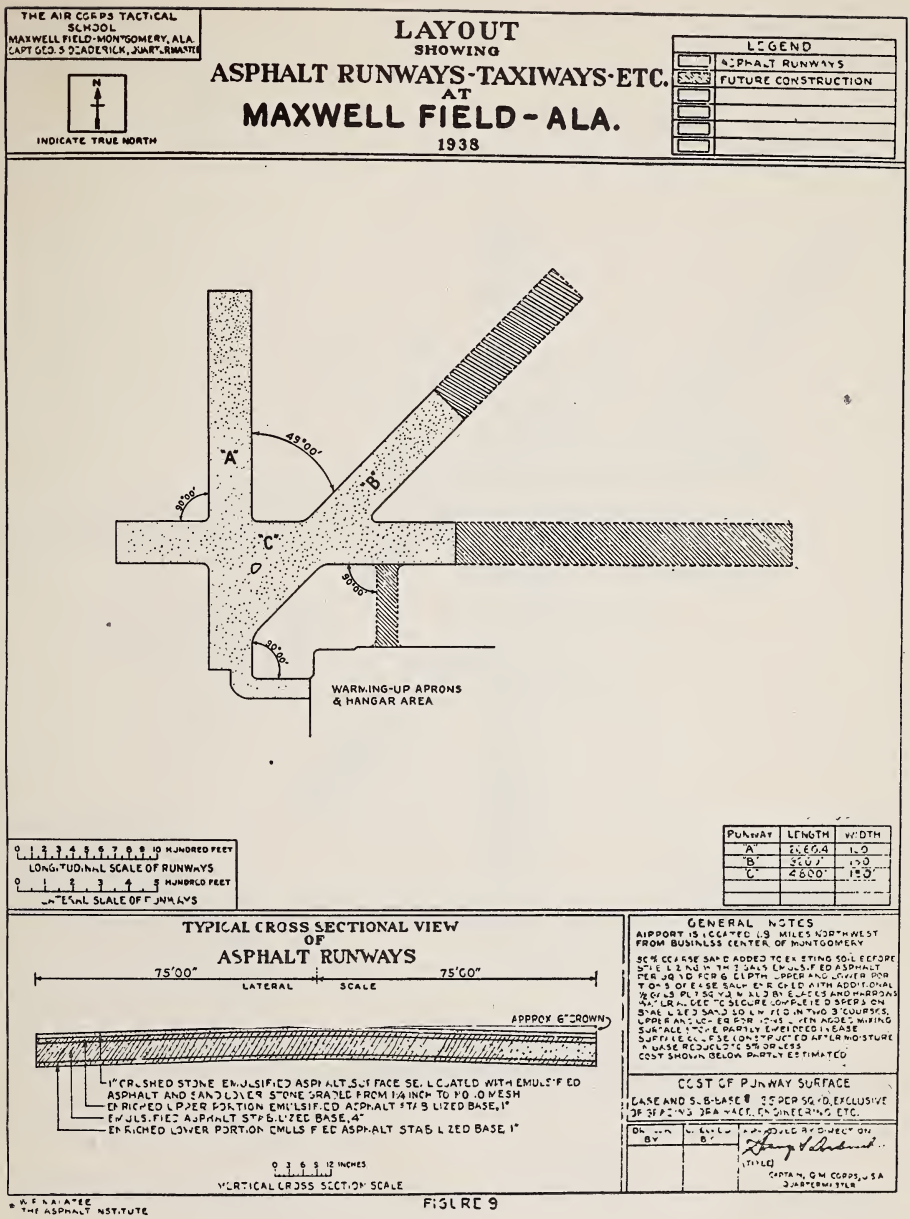


FIGURE 9

The runways there were partially stabilized with SC-3, later supplanted by cutback asphalt, RC-1. This change was made because the slow-curing type entailed an undue delay before the surface of the runways would be ready to use.

The Municipal Airport (also at Pensacola) (Fig. 10) was constructed by stabilizing the local sand found in runway areas, using cutback type of asphalt. The nomenclature employed in describing this work also reveals the desirability of developing a more generally understood common term for identifying such mixtures. This is identified as sand-asphalt but which is also a form of asphalt stabilization. Generally speaking, asphalt stabilization is suitable only for base constructon and is not desirable for wearing surfaces as it is generally too friable to withstand the abrasive action of traffic. An exception to this generalization is noted in asphalt stabiliza-

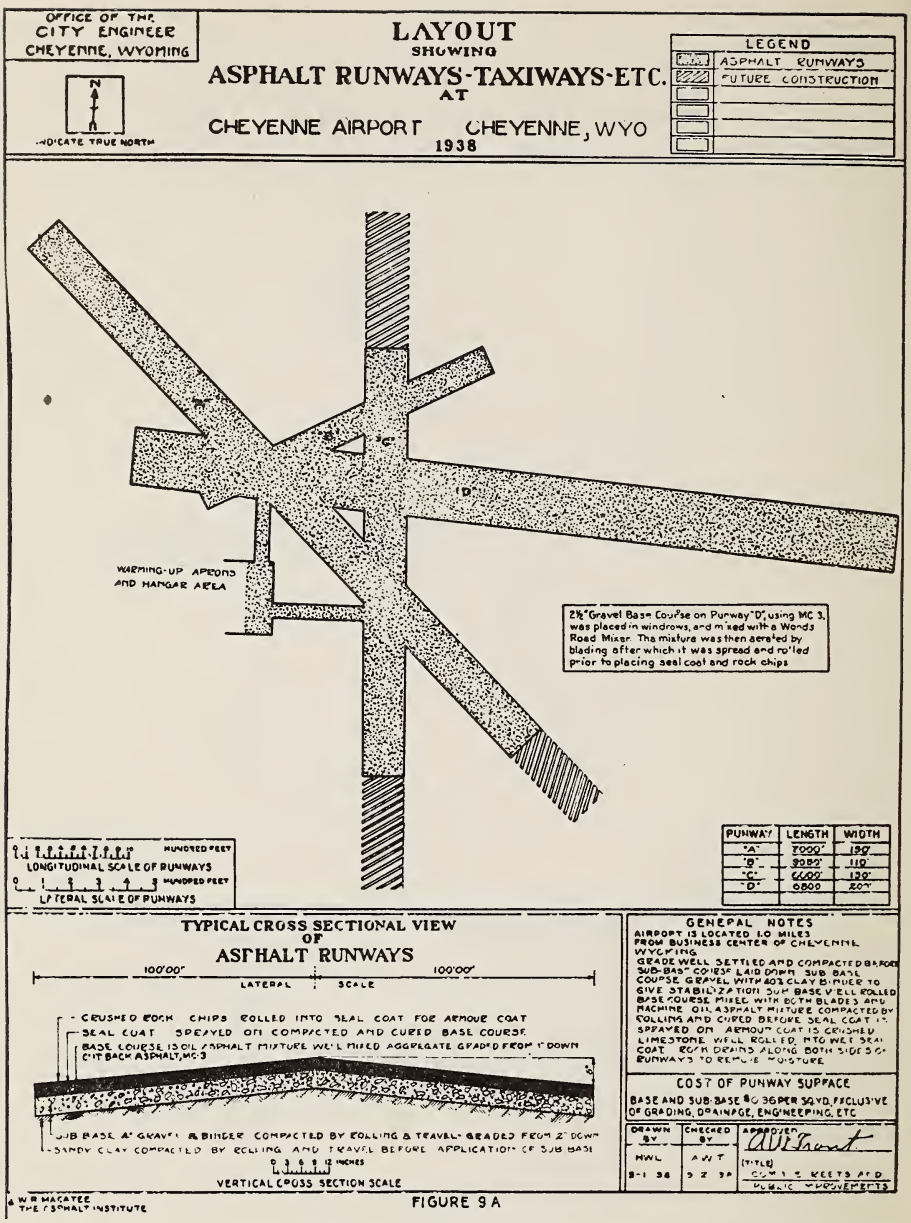

OFFICE OF THE CITY ENGINEER CHEYENNE, WYOMING
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
CHEYENNE AIRPORT CHEYENNE, WYO
1938
INDICATE TRUE NORTH
LEGEND
ASPHALT RUNWAYS
FUTURE CONSTRUCTION
WARMING-UP APRONS AND HANGAR AREA
2½" Gravel Base Course on Runway "D", using MC 3, was placed in windrows, and mixed with a Wonds Road Mixer. The mixture was then aerated by blading after which it was spread and rolled prior to placing seal coat and rock chips
RUNWAY | LENGTH | WIDTH
"A" 7000' 150'
"B" 9252' 110'
"C" 6600' 120'
"D" 6800 200'
HUNDRED FEET
LONGITUDINAL SCALE OF RUNWAYS
HUNDRED FEET
LATERAL SCALE OF RUNWAYS
TYPICAL CROSS SECTIONAL VIEW OF ASPHALT RUNWAYS
100'00" LATERAL SCALE 100'00"
CRUSHED ROCK CHIPS ROLLED INTO SEAL COAT FOR ARMOUR COAT
SEAL COAT SPRAYED ON COMPACTED AND CURED BASE COURSE
BASE COURSE IS OIL ASPHALT MIXTURE WELL MIXED AGGREGATE GRADED FROM 1" DOWN CUT BACK ASPHALT, MC-3
SUB BASE 4" GRAVEL & BINDER COMPACTED BY ROLLING & TRAVEL GRADED FROM 2" DOWN
SANDY CLAY COMPACTED BY ROLLING AND TRAVEL BEFORE APPLICATION OF SUB BASE
12 INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 10 MILES FROM BUSINESS CENTER OF CHEYENNE, WYOMING
GRADE WELL SETTLED AND COMPACTED BEFORE SUB-BASE COURSE LAID DOWN. SUB BASE COURSE GRAVEL WITH 40% CLAY BINDER TO GIVE STABILIZATION. SUB BASE WELL ROLLED BASE COURSE MIXED WITH BOTH BLADES AND MACHINE OIL ASPHALT MIXTURE COMPACTED BY ROLLING AND CURED BEFORE SEAL COAT IS SPRAYED ON. ARMOUR COAT IS CRUSHED LIMESTONE WELL ROLLED INTO WET SEAL COAT. ROCK DRIVEN ALONG BOTH SIDES OF RUNWAYS TO REMOVE MOISTURE
COST OF RUNWAY SURFACE
BASE AND SUB-BASE @ 36 PER SQ YD. EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC
DRAWN BY HWL 8-1 36
CHECKED BY AWT 9 2 36
APPROVED (TITLE) CITY ENGINEER & SUPT OF PUBLIC IMPROVEMENTS
© W.P. MACATEE THE ASPHALT INSTITUTE

FIGURE 9 A

tion work where the aggregate is principally sand such as found in Charleston (Fig. 8), Pensacola (Fig. 10) and Reeves Field (Fig. 2).

In reviewing the reports received in which asphalt stabilization work has been carried on, it will be observed that this fast-developing use for asphalt has included the slow-curing type, medium-curing type, rapid-curing type and the emulsified type as stabilizing agents.

CUT-OFF SIDE WALLS

Drawings revealing the cross-sectional views of the Topeka, Kansas Airport (Fig. 11) and the Los Angeles Municipal Airport (Fig. 12) bring out very clearly the trend toward using asphaltic mixtures for producing cut-off or side walls at the sides and ends of runways. The purpose of these cut-off walls is obvious, that is, to prevent water from adjacent,

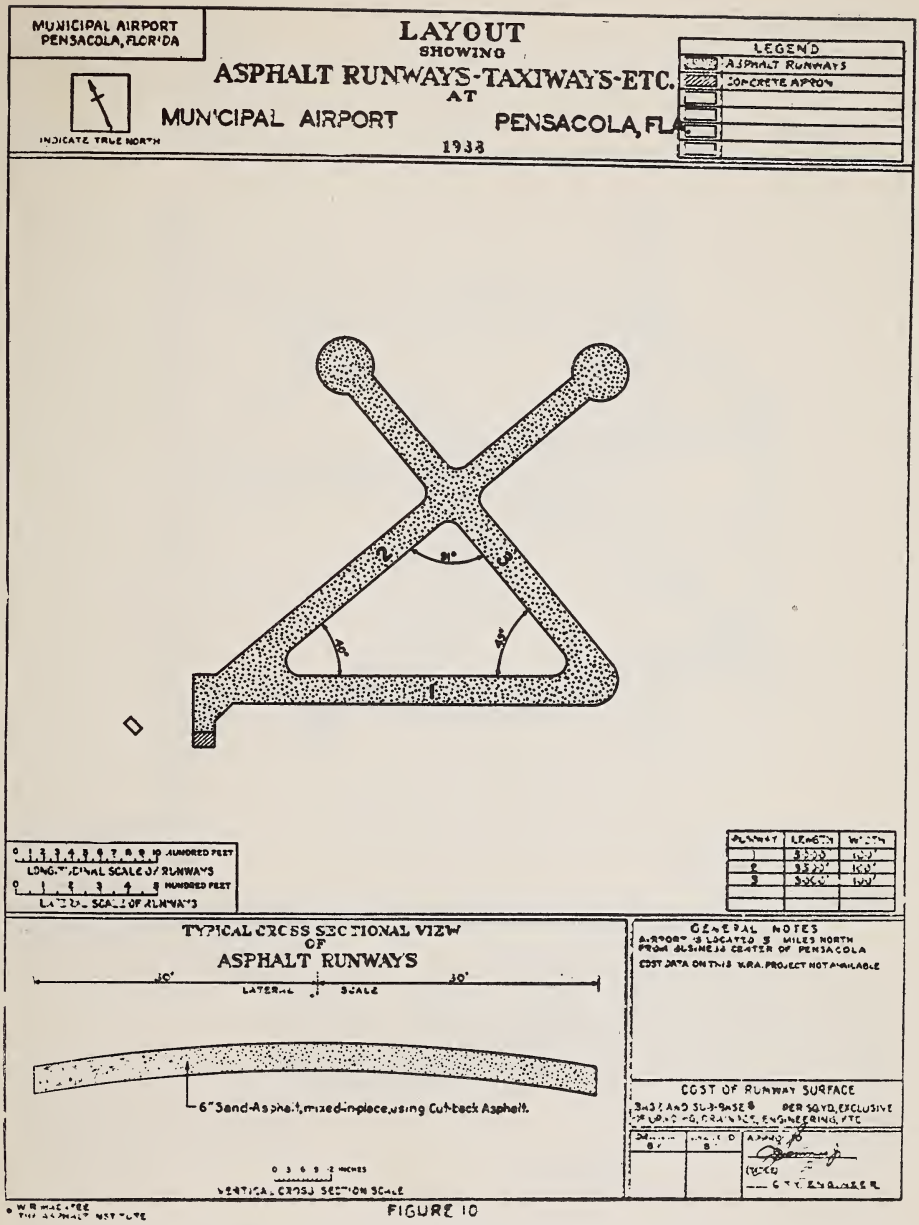


FIGURE 10

unpaved areas from finding its way horizontally under the runway surface structure—especially at the edges.

The engineers at Los Angeles Municipal Airport (Fig. 12) employed the same type of mixture in the cut-off walls as was used in the base course, whereas, the engineering authorities at Topeka (Fig. 11) used coarse stone graded from 2½″ down to 1″, merely coating the stone with asphalt to lightly bind it together, thus permitting infiltration of water from the surface to be guided down and into clay-tile drains which safely carry off the water and prevent it from destroying the integrity of the subgrade.

EXTENSIONS ALREADY PLANNED

Among the outstanding air field developments revealed by a large

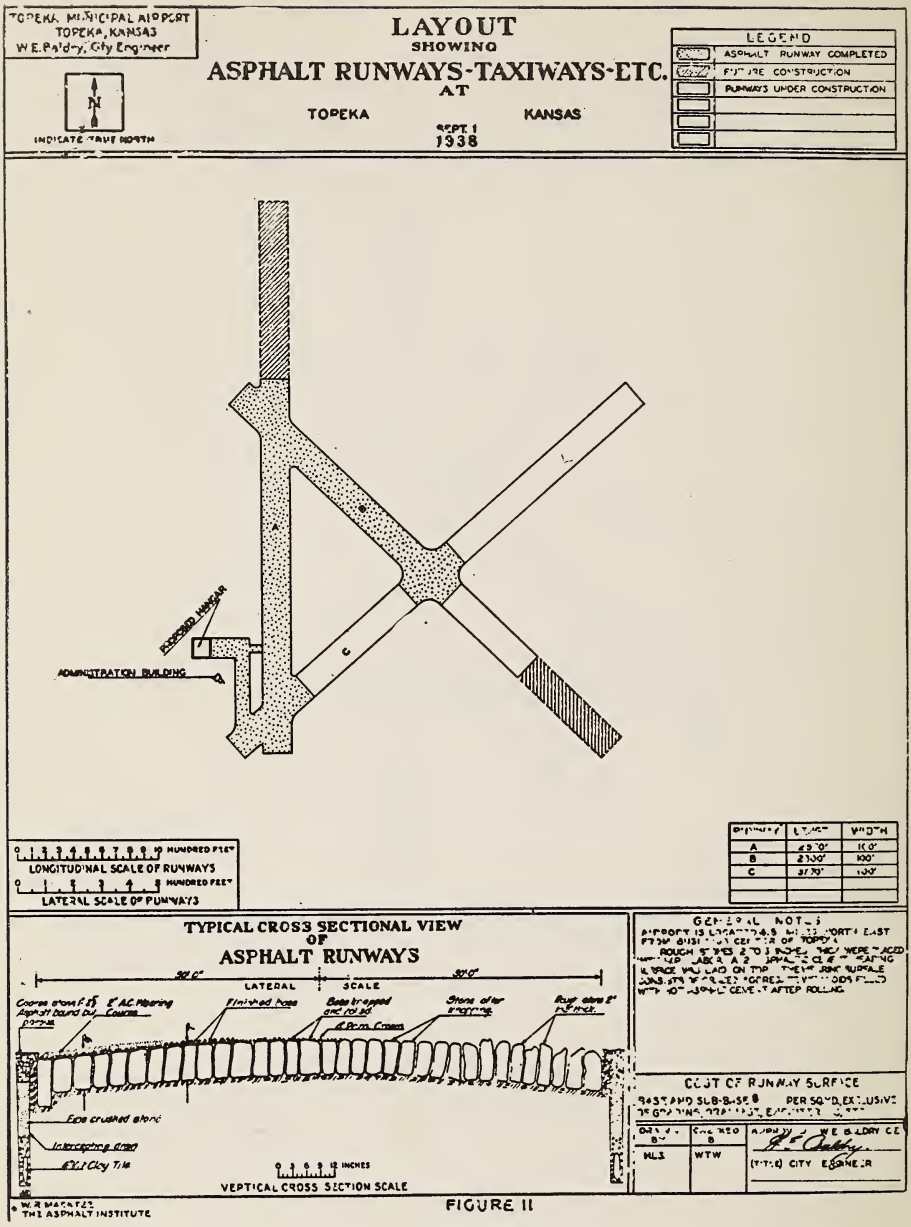


FIGURE 11

number of replies to my questionnaire was the fact that engineers responsible for airport operations have, generally speaking, anticipated the need for early and large extensions to present-day facilities. This realization of needs which are in the immediate offing are revealed in the drawing which describes the Los Angeles Municipal Airport (Fig. 12). By the way, that enterprising metropolitan city on the Pacific Coast not only is planning large extensions to its Municipal Airport (Fig. 12) but it also operates another important airport in the San Pedro section of the city (Fig. 2).

The proposed extensions for which plans are being made by many airport authorities are not limited to the big cities, but engineers in the smaller communities are equally alert to the need for providing plans for extending airports in their cities. This is brought out quite clearly in the descriptive drawing of the airport at Wichita Falls, Texas (Fig. 13).

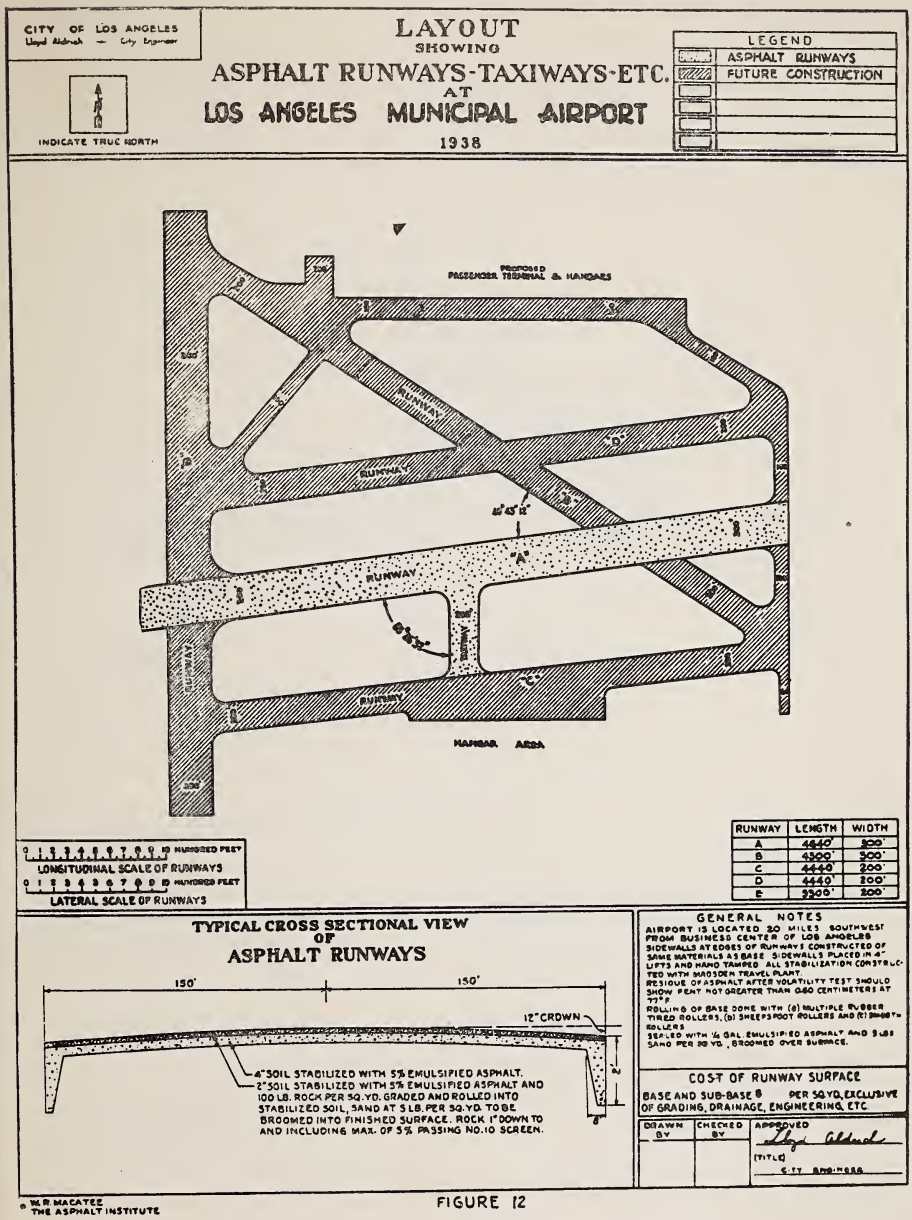

CITY OF LOS ANGELES
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
LOS ANGELES MUNICIPAL AIRPORT
1938
INDICATE TRUE NORTH
LEGEND
ASPHALT RUNWAYS
FUTURE CONSTRUCTION
PROPOSED PASSENGER TERMINAL & HANGARS
RUNWAY
HANGAR AREA
LONGITUDINAL SCALE OF RUNWAYS
LATERAL SCALE OF RUNWAYS
RUNWAY | LENGTH | WIDTH
A 4440' 300'
B 4300' 300'
C 4440' 200'
D 4440' 200'
E 3300' 200'
TYPICAL CROSS SECTIONAL VIEW OF ASPHALT RUNWAYS
150'
150'
12" CROWN
4" SOIL STABILIZED WITH 5% EMULSIFIED ASPHALT.
2" SOIL STABILIZED WITH 5% EMULSIFIED ASPHALT AND 100 LB. ROCK PER SQ. YD. GRADED AND ROLLED INTO STABILIZED SOIL, SAND AT 5 LB. PER SQ. YD. TO BE BROOMED INTO FINISHED SURFACE. ROCK 1" DOWN TO AND INCLUDING MAX. OF 5% PASSING NO. 10 SCREEN.
GENERAL NOTES
AIRPORT IS LOCATED 20 MILES SOUTHWEST FROM BUSINESS CENTER OF LOS ANGELES
SIDEWALLS AT EDGES OF RUNWAYS CONSTRUCTED OF SAME MATERIALS AS BASE SIDEWALLS PLACED IN 4" LIFTS AND HAND TAMPED. ALL STABILIZATION CONSTRUCTED WITH HVEEM TRAVEL PLANT.
RESIDUE OF ASPHALT AFTER VOLATILITY TEST SHOULD SHOW PENT NOT GREATER THAN 060 CENTIMETERS AT 77°F
ROLLING OF BASE DONE WITH (a) MULTIPLE RUBBER TIRED ROLLERS, (b) SHEEPSFOOT ROLLERS AND (c) SMOOTH ROLLERS
SEALED WITH ¼ GAL EMULSIFIED ASPHALT AND 3 LBS SAND PER SQ YD., BROOMED OVER SURFACE.
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $ PER SQ YD, EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
(TITLE)
CITY ENGINEER
W. R. MACATEE
THE ASPHALT INSTITUTE

FIGURE 12

SIZE OF SURFACED AREA AT AIR FIELDS

The average surfaced area at airports illustrated in this paper is about 175,000 square yards which is equivalent to nearly 17 miles of 18-foot roadway. Whereas, 175,000 square yards is the average, the size of the community in which the airport is located governs the area to be surfaced. For instance, the big Chicago Municipal Airport (Fig. 3) has surfaced area equivalent to 104 miles of 18-foot roadway.

ABSENCE OF CURLING-FATIGUE IN ASPHALT RUNWAYS

One engineer brought out quite interestingly one of the advantages found in asphalt for airport surfaces. This comment focused attention on the fact that flexible types of runway surfaces are not subjected to the curling action by change of air temperatures, and are therefore not

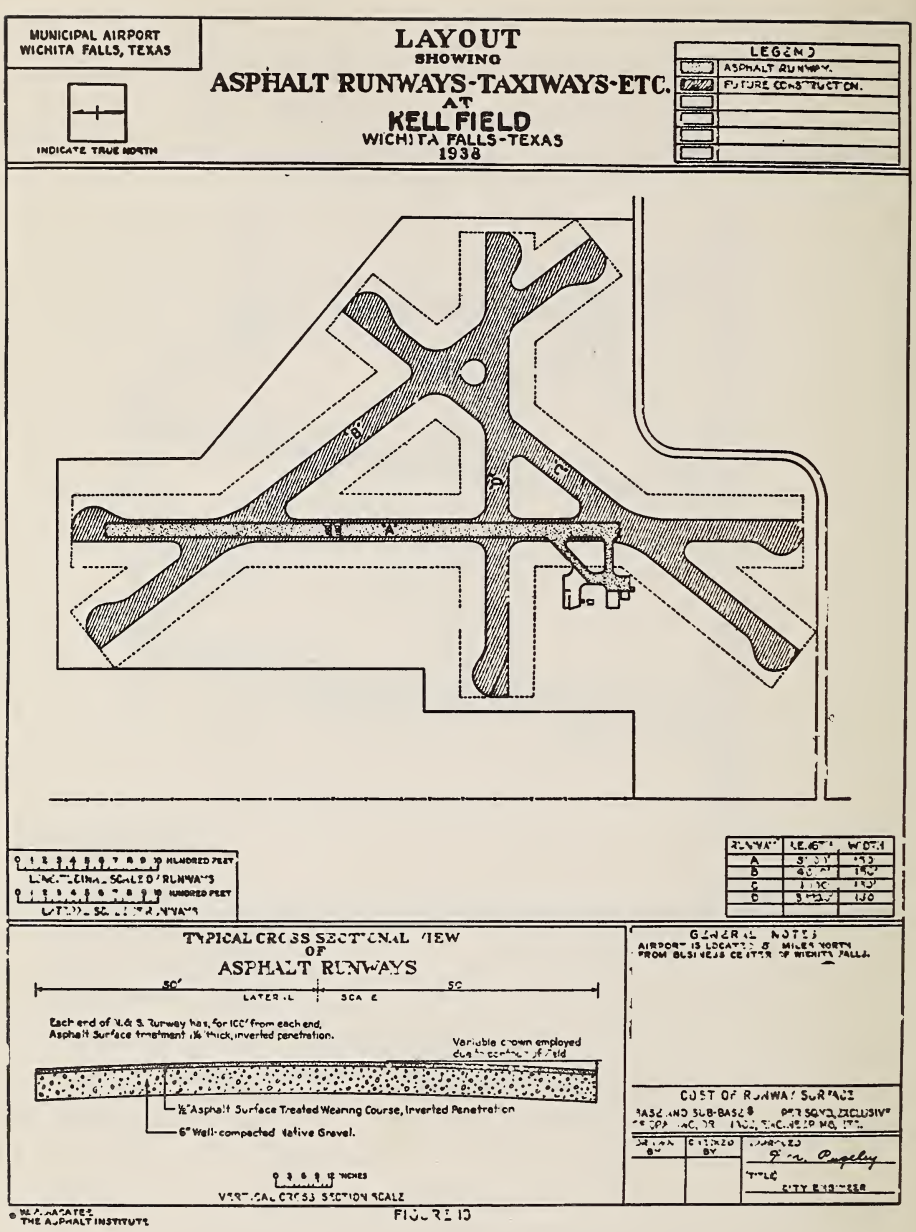


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FIGURE 10

subject to the destructive forces of fatigue from that cause. Fatigue occasioned by the action of traffic, such as we understand that term in relation to highways, is of no importance in runways, for the reason that traffic on runways is a negligible factor; the identically same lanes of traffic being used relatively seldom.

STAGE-CONSTRUCTION AS REQUIRED

Another feature pointing to the desirability of using asphalt in runway construction was brought out by an airport official in one of the eastern cities to the effect that it serves advantageously because of the facility and economy with which it can be strengthened by additional layers from time to time as required by increased weight of planes using airports, or for the periodic surface renewal to replace wear.

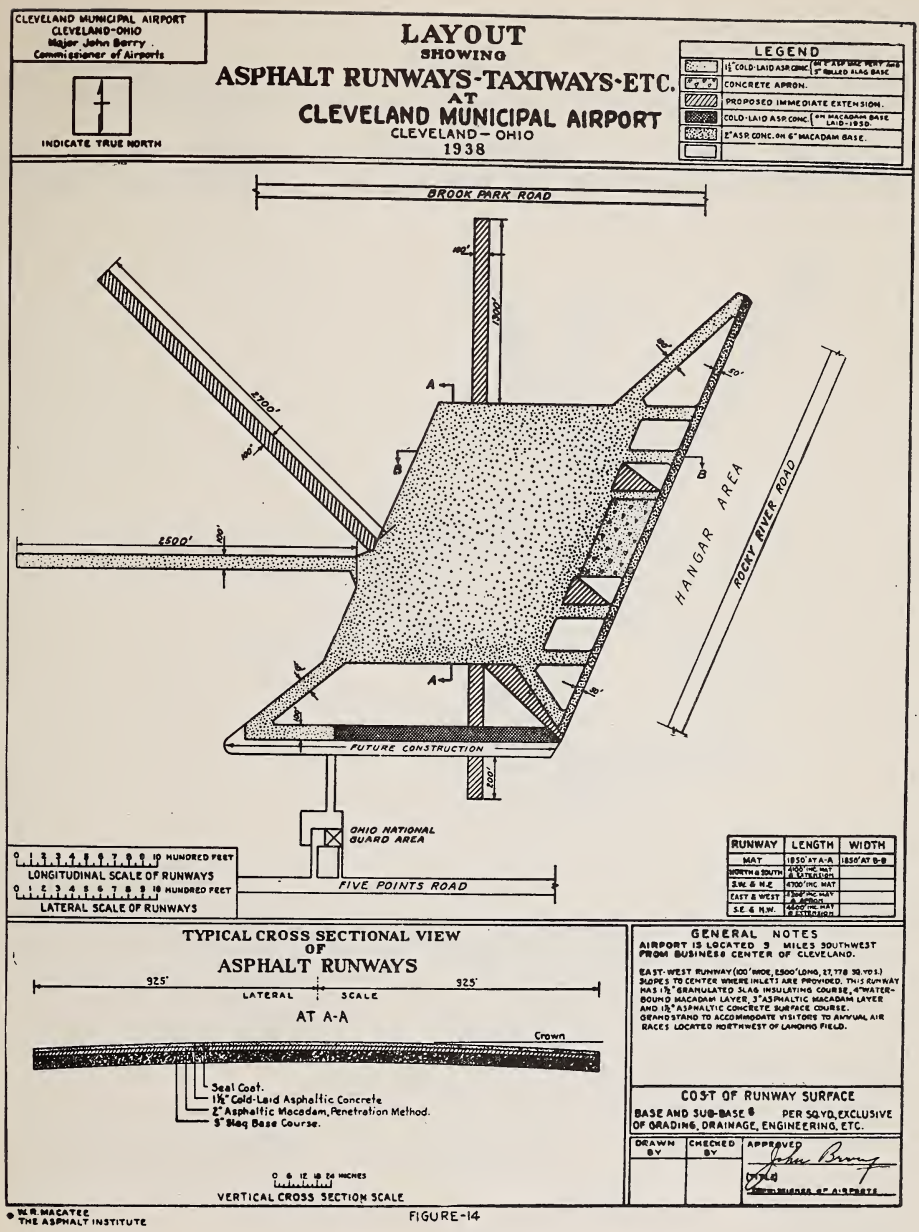
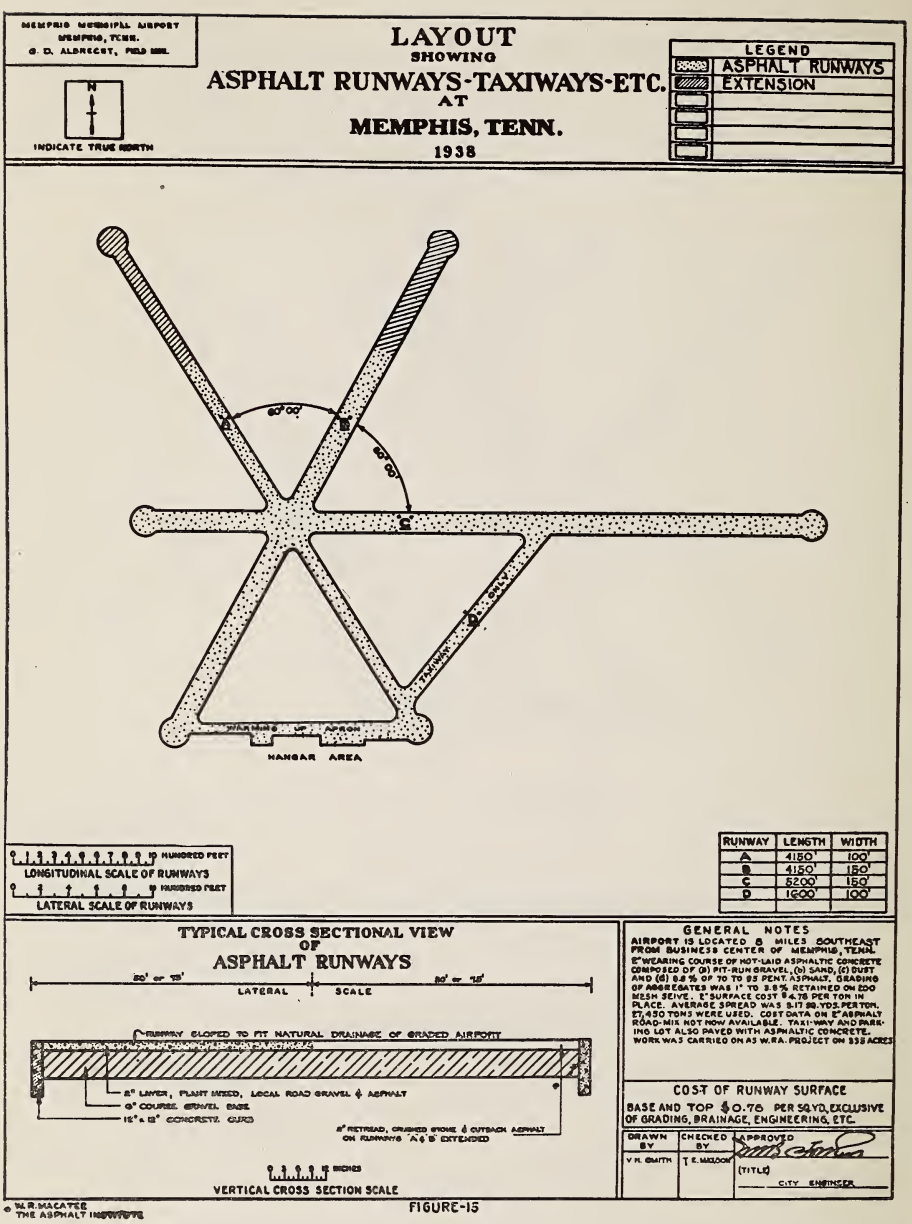

CLEVELAND MUNICIPAL AIRPORT
CLEVELAND-OHIO
Major John Berry
Commissioner of Airports
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
CLEVELAND MUNICIPAL AIRPORT
CLEVELAND — OHIO
1938
LEGEND
1½" COLD-LAID ASP. CONC.
CONCRETE APRON.
PROPOSED IMMEDIATE EXTENSION.
COLD-LAID ASP. CONC.
2" ASP. CONC. ON 6" MACADAM BASE.
BROOK PARK ROAD
2700'
2500'
HANGAR AREA
ROCKY RIVER ROAD
FUTURE CONSTRUCTION
OHIO NATIONAL GUARD AREA
FIVE POINTS ROAD
LONGITUDINAL SCALE OF RUNWAYS
LATERAL SCALE OF RUNWAYS
RUNWAY | LENGTH | WIDTH
MAT | 1850' AT A-A | 1850' AT B-B
NORTH & SOUTH | 4700' INC. MAT & EXTENSION |
S.W. & N.E. | 4700' INC. MAT |
EAST & WEST | 3500' INC. MAT & EXTENSION |
S.E. & N.W. | 4600' INC. MAT & EXTENSION |
TYPICAL CROSS SECTIONAL VIEW OF ASPHALT RUNWAYS
925'
LATERAL SCALE
925'
AT A-A
Crown
Seal Coat.
1½" Cold-Laid Asphaltic Concrete
2" Asphaltic Macadam, Penetration Method.
5" Slag Base Course.
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 9 MILES SOUTHWEST FROM BUSINESS CENTER OF CLEVELAND.
EAST-WEST RUNWAY (100' WIDE, 2500' LONG, 27,778 SQ. YDS.) SLOPES TO CENTER WHERE INLETS ARE PROVIDED. THIS RUNWAY HAS 1½" GRANULATED SLAG INSULATING COURSE, 4" WATER-BOUND MACADAM LAYER, 3" ASPHALTIC MACADAM LAYER AND 1½" ASPHALTIC CONCRETE SURFACE COURSE.
GRAND STAND TO ACCOMMODATE VISITORS TO ANNUAL AIR RACES LOCATED NORTHWEST OF LANDING FIELD.
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $ PER SQ. YD. EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
John Berry
COMMISSIONER OF AIRPORTS
W. R. MACATEE
THE ASPHALT INSTITUTE

FIGURE-14

WIDTHS OF LANES OF CONSTRUCTION

The replies further showed that there was no unanimity of practice with regard to width of lanes of construction. It was, however, made fairly evident that the maximum width—therefore, presumably the most practicable and economical—was 20 feet. In the case of construction of either bases or surface courses by means of traveling mixing-plants, the usual width of construction of lanes was found to be 8 to 11 feet.

Where local soils were utilized for base courses or base and surface courses (such as sand-asphalt), it was found that the usual practice is to construct runways in alternate lanes. The practice of constructing bases in alternate lanes has especially practical value where they are stabilized with asphalt in two courses, as the areas adjacent to the lane under construction provide space for the storage of mixtures which are


MEMPHIS MEMORIAL AIRPORT
MEMPHIS, TENN.
G. D. ALBRECHT, FIELD ENG.
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
MEMPHIS, TENN.
1938
INDICATE TRUE NORTH
LEGEND
ASPHALT RUNWAYS
EXTENSION
60°00'
60°00'
TAXIWAY
GRAVEL
MARKING UP APRON
HANGAR AREA
HUNDRED FEET
LONGITUDINAL SCALE OF RUNWAYS
HUNDRED FEET
LATERAL SCALE OF RUNWAYS
RUNWAY | LENGTH | WIDTH
A | 4150' | 100'
B | 4150' | 150'
C | 5200' | 150'
D | 1600' | 100'
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
50' or 75'
LATERAL
SCALE
50' or 75'
RUNWAY SLOPED TO FIT NATURAL DRAINAGE OF GRADED AIRPORT
2" LAYER, PLANT MIXED, LOCAL ROAD GRAVEL & ASPHALT
6" COURSE GRAVEL BASE
12"x 8" CONCRETE CURB
6" RETREAD, CRUSHED STONE & OUTBACH ASPHALT
ON RUNWAYS 'A & B' EXTENDED
INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 8 MILES SOUTHEAST FROM BUSINESS CENTER OF MEMPHIS, TENN.
2" WEARING COURSE OF HOT-LAID ASPHALTIC CONCRETE COMPOSED OF (a) PIT-RUN GRAVEL, (b) SAND, (c) DUST AND (d) 6.8 % OF 70 TO 85 PENT. ASPHALT. GRADING OF AGGREGATES WAS 1" TO 8 8% RETAINED ON 200 MESH SEIVE. 2" SURFACE COST $4.78 PER TON IN PLACE. AVERAGE SPREAD WAS 5.17 SQ. YDS. PER TON. 27,430 TONS WERE USED. COST DATA ON 2" ASPHALT ROAD-MIX NOT NOW AVAILABLE. TAXI-WAY AND PARKING LOT ALSO PAVED WITH ASPHALTIC CONCRETE. WORK WAS CARRIED ON AS W.P.A. PROJECT ON 338 ACRES
COST OF RUNWAY SURFACE
BASE AND TOP $0.75 PER SQ.YD. EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
V.H. SMITH
CHECKED BY
T.E. MILLSOP
APPROVED
(TITLE)
CITY ENGINEER
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THE ASPHALT INSTITUTE

FIGURE-15

made first but laid aside—for future use in the upper layer—while the lower layer is mixed, cured and rolled.

LANDING MATS

One of the important air fields in the country is located at Cleveland (Fig. 14). In addition to being distinctive because of heavy commercial air traffic, it also is the scene of the country's annual air races, the classic in aviation sports competition. Aside from these distinctions, it is constructed in a manner which is unusual, whereby the conventional type of runways is combined with a huge landing mat, having dimensions of 1850 feet by 1850 feet, equal to nearly 400,000 square yards. This is in addition to the conventional type of runways which are appendages to the mat as shown in Fig. 14.

Operators of airports recognize in the landing mat-runway type fea-

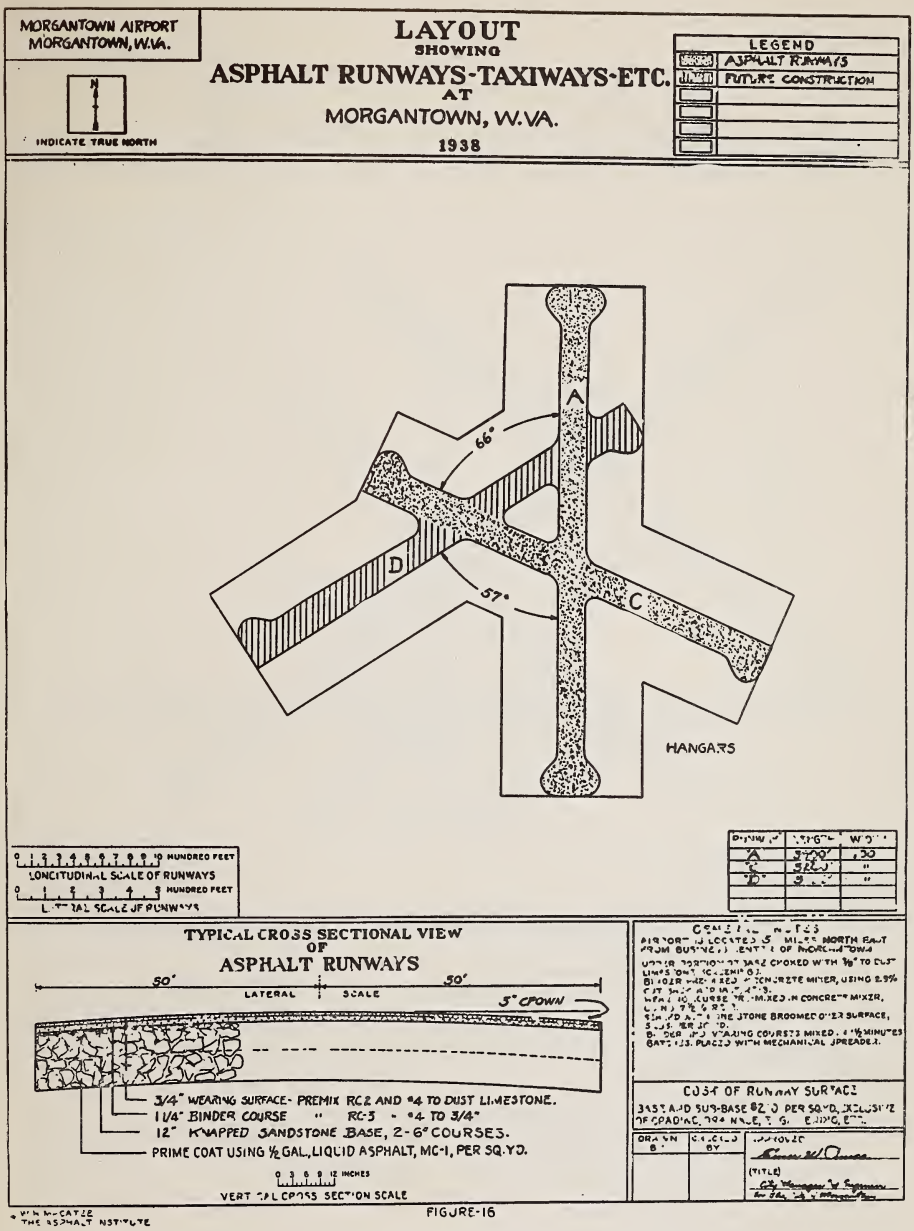


FIGURE-16

tures which commend it, especially in blind landing which aviation is perfecting. The Saturday Evening Post in a recent issue treating the general subject of aviation favorably commented on this particular type of runway.

It will be observed in the cross-sectional view of the drawing describing Cleveland's air field that the principle of enriching the wearing surface was carried out by providing a seal coat on the 1½″ cold-laid plant-mix type of asphaltic concrete. It will also be observed that the asphaltic concrete was supported by a 5″ slag base course with an intermediate 2″ course of asphaltic macadam constructed by the penetration method.

TREND TO USE OF LIQUID ASPHALT

A pronounced trend toward the use of cutback asphalt, such as the

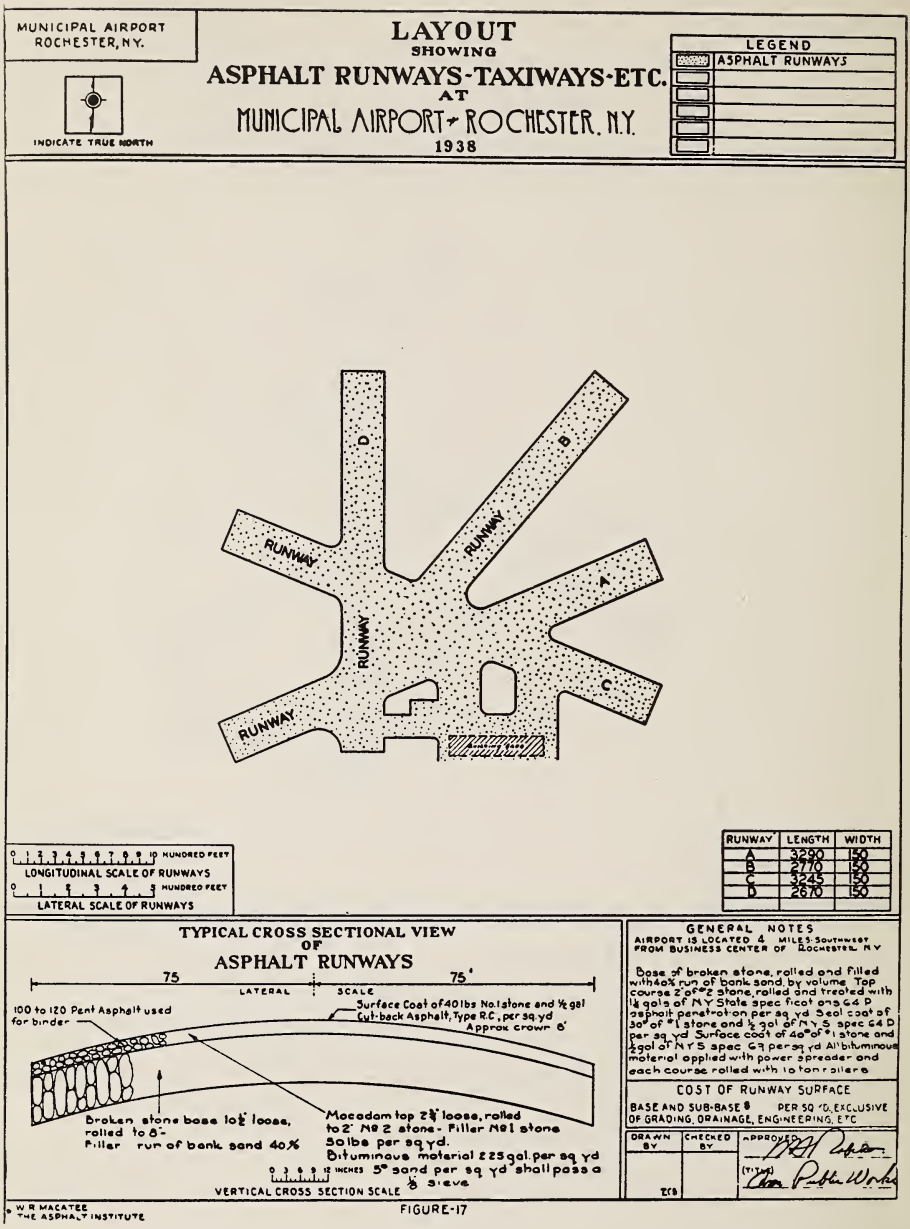

MUNICIPAL AIRPORT
ROCHESTER, N.Y.
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
MUNICIPAL AIRPORT - ROCHESTER, N.Y.
1938
LEGEND
ASPHALT RUNWAYS
RUNWAY
RUNWAY
RUNWAY
RUNWAY
A
B
C
D
LONGITUDINAL SCALE OF RUNWAYS
LATERAL SCALE OF RUNWAYS
RUNWAY LENGTH WIDTH
A 3290 150
B 2770 150
C 3245 150
D 2670 150
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
75
LATERAL SCALE
75'
100 to 120 Pent Asphalt used for binder
Surface Coat of 40 lbs No. 1 stone and ½ gal Cut-back Asphalt, Type R.C., per sq yd
Approx crown 6"
Broken stone base 10" loose, rolled to 8"
Filler run of bank sand 40%
Macadam top 2½" loose, rolled to 2" No 2 stone - Filler No 1 stone
50 lbs per sq yd.
Bituminous material 2.25 gal. per sq yd
5% sand per sq yd shall pass a ⅜ sieve
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 4 MILES SOUTHWEST FROM BUSINESS CENTER OF ROCHESTER, N.Y.
Base of broken stone, rolled and filled with 40% run of bank sand by volume. Top course 2" of 2" stone, rolled and treated with ½ gal/s of N.Y. State spec ficat one 64 D asphalt penetration per sq yd Seal coat of 30" of #1 stone and ½ gal of N.Y.S. spec 64 D per sq yd Surface coat of 40 lbs of #1 stone and ½ gal of N.Y.S. spec 69 per sq yd All bituminous material applied with power spreader and each course rolled with 10 ton rollers
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $ PER SQ YD, EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC
DRAWN BY
CHECKED BY
APPROVED
(TITLE)
W.R. MACATEE
THE ASPHALT INSTITUTE

FIGURE-17

naphtha type rapid-curing products and the kerosene type medium-curing grades, was revealed in the replies received. This trend is made clear in the case of the extensions of Runways "A" and "B" installed at Memphis (Fig. 15). The drawing shows that a cutback asphalt was used for the second surfacing program in that city. Another revelation of this trend is shown in Fig. 16, descriptive of the Morgantown, West Virginia Airport. This is probably due to a number of factors, not least of which is the desire to use a base stock asphalt of relatively high penetration.

ASPHALT PRIMING MATERIAL

The drawing which reveals details used in constructing the Morgantown, West Virginia Airport (Fig. 16) illustrates the use of liquid asphalt, MC-1, for priming purposes. A prepared base, such as rolled stone, gravel

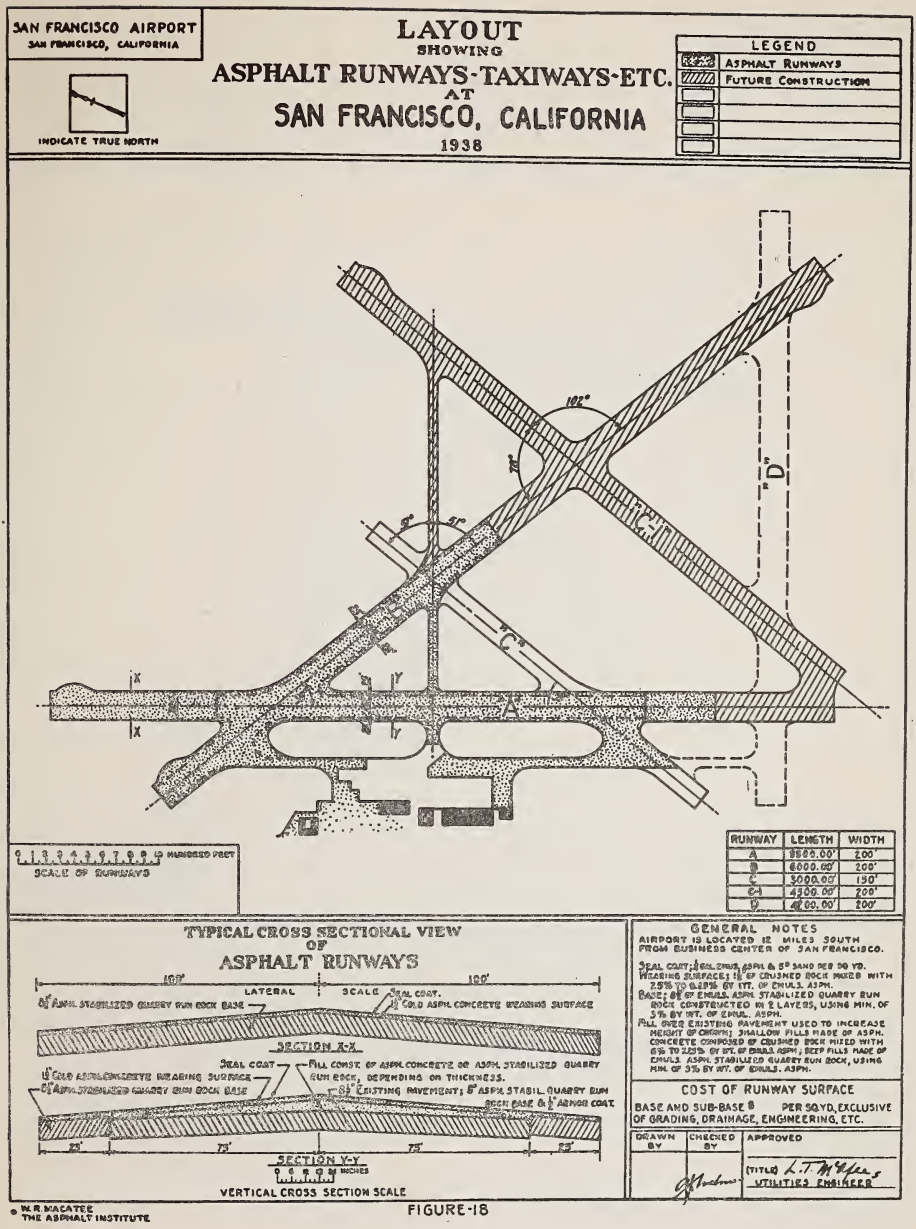

SAN FRANCISCO AIRPORT
SAN FRANCISCO, CALIFORNIA
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
SAN FRANCISCO, CALIFORNIA
1938
LEGEND
ASPHALT RUNWAYS
FUTURE CONSTRUCTION
102°
78°
"D"
"C"
"C-1"
RUNWAY | LENGTH | WIDTH
A | 9800.00' | 200'
B | 6000.00' | 200'
C | 3000.00' | 150'
C-1 | 4500.00' | 200'
D | 4000.00' | 200'
SCALE OF RUNWAYS
TYPICAL CROSS SECTIONAL VIEW OF ASPHALT RUNWAYS
LATERAL
SCALE
SEAL COAT.
COLD ASPH. CONCRETE WEARING SURFACE
SECTION X-X
SEAL COAT
FILL CONSIS OF ASPH. CONCRETE OR ASPH. STABILIZED QUARRY RUN ROCK, DEPENDING ON THICKNESS.
EXISTING PAVEMENT; 6" ASPH. STABIL. QUARRY RUN ROCK BASE & 1" ARMOR COAT
SECTION Y-Y
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 12 MILES SOUTH FROM BUSINESS CENTER OF SAN FRANCISCO.
SEAL COAT; 1/2 GAL. EMUL. ASPH. & 8# SAND PER SQ YD.
WEARING SURFACE; 1" OF CRUSHED ROCK MIXED WITH 7.5% TO 8.25% BY WT. OF EMULS. ASPH.
BASE; 6" OF EMULS. ASPH. STABILIZED QUARRY RUN ROCK CONSTRUCTED IN 2 LAYERS, USING MIN. OF 5% BY WT. OF EMUL. ASPH.
FILL OVER EXISTING PAVEMENT USED TO INCREASE HEIGHT OF CROWN; SHALLOW FILLS MADE OF ASPH. CONCRETE COMPOSED OF CRUSHED ROCK MIXED WITH 6% TO 7.25% BY WT. OF EMULS ASPH; DEEP FILLS MADE OF EMULS. ASPH. STABILIZED QUARRY RUN ROCK, USING MIN. OF 5% BY WT. OF EMULS. ASPH.
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $ PER SQ YD, EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
(TITLE) L. T. McGee
UTILITIES ENGINEER
W. R. MACATEE
THE ASPHALT INSTITUTE

FIGURE-18

and sand-clay, should be treated with an asphalt primer before road-mix, or plant-produced surfaces are placed. Medium-curing liquid asphalt, MC-1, such as used at Morgantown is suitable for priming such bases unless they are of a very porous, open texture. In the latter case, the use of MC-2 is indicated. MC-1 is a low-viscosity material which is readily and completely absorbed by the surface and has proved extremely satisfactory as a primer in airport and highway construction.

PARKING LOTS AT AIRPORTS

A modern, adequate parking lot for automobiles is an important adjunct to provide at airports. It will be observed in Fig. 15, descriptive of the Memphis Airport, that hot-laid asphaltic concrete was used in paving the parking lot areas.

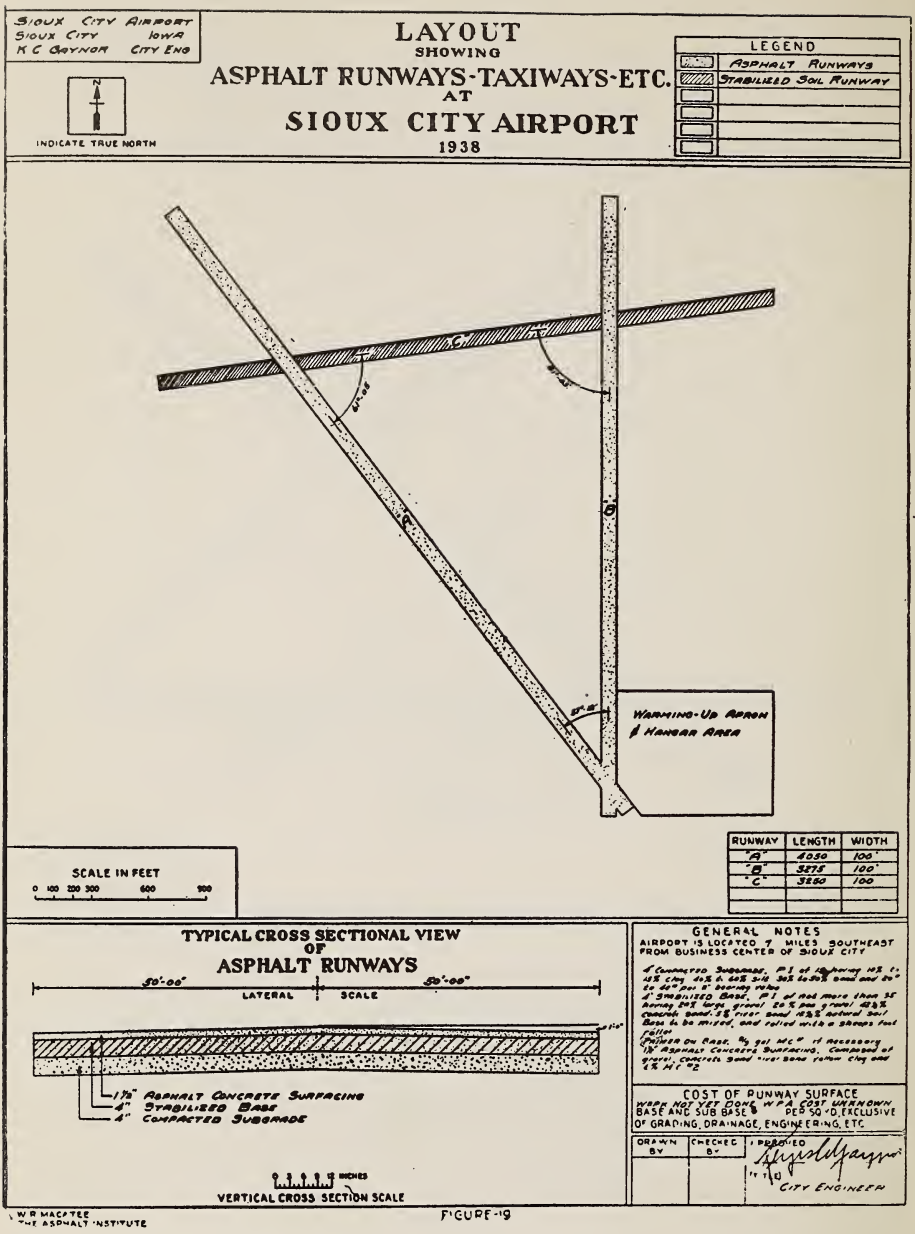
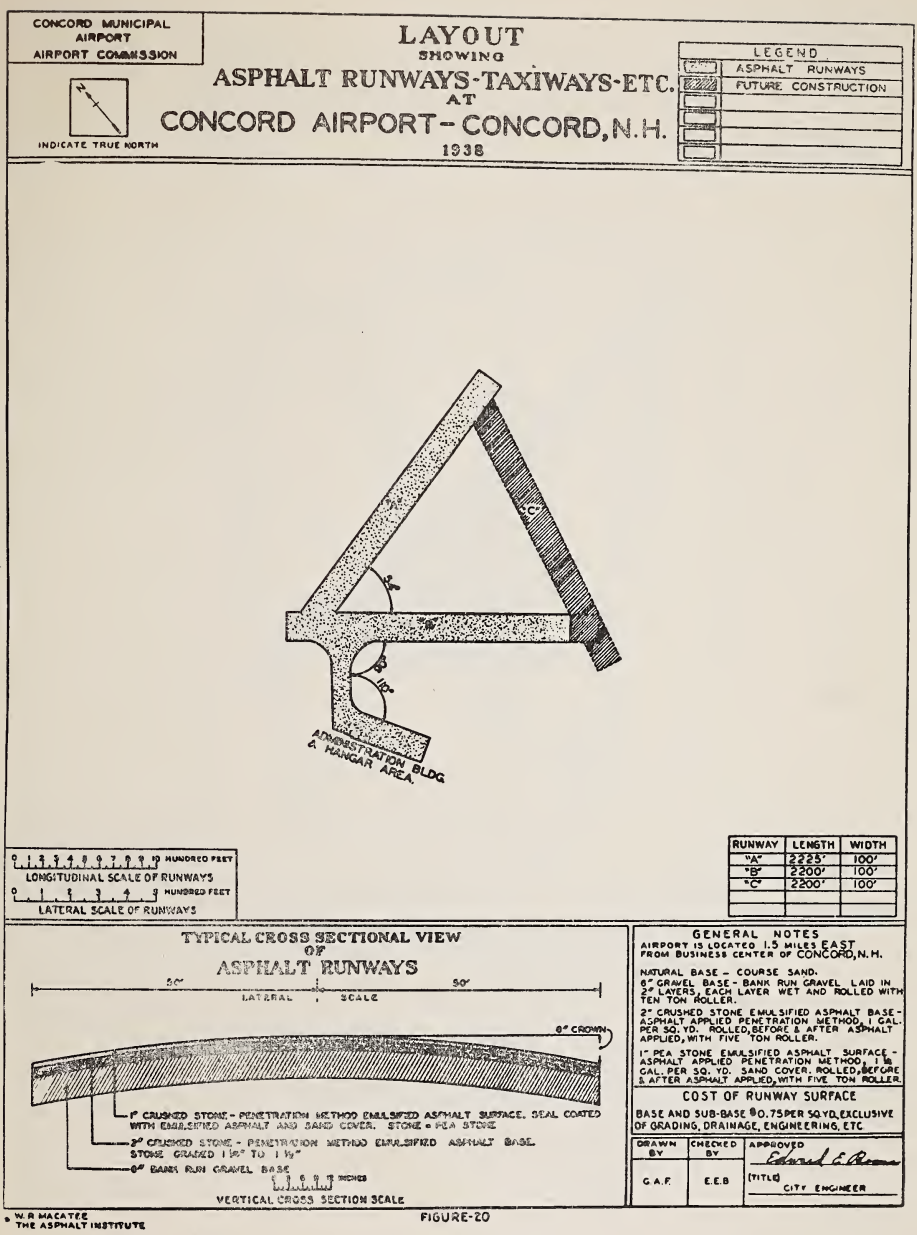


FIGURE-19

Aprons and hangar floors preferably should be paved with an asphalt mixture of somewhat higher stability than is required on taxiways and runways. Asphalt paving blocks are ideally suited for hangar floors, especially where mechanical servicing of planes is carried on. Asphalt blocks also serve equally well on aprons adjoining hangars. If a monolithic type of surfacing is desired, asphaltic macadam of the penetration type or plant-mix type of asphaltic concrete is indicated for hangar floors and aprons.

RUNWAYS OF STURDY PENETRATION MACADAM

Descriptive drawing of the Rochester, New York Airport (Fig. 17) illustrates a sturdy type of construction which has long been popular in the highway field in New York State, New England, Oregon and other


CONCORD MUNICIPAL AIRPORT
AIRPORT COMMISSION
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
CONCORD AIRPORT - CONCORD, N.H.
1938
LEGEND
ASPHALT RUNWAYS
FUTURE CONSTRUCTION
"A"
"B"
"C"
34°
45°
102°
ADMINISTRATION BLDG. & HANGAR AREA
HUNDRED FEET
LONGITUDINAL SCALE OF RUNWAYS
LATERAL SCALE OF RUNWAYS
RUNWAY | LENGTH | WIDTH
"A" | 2225' | 100'
"B" | 2200' | 100'
"C" | 2200' | 100'
TYPICAL CROSS SECTIONAL VIEW OF ASPHALT RUNWAYS
50'
LATERAL SCALE
50'
6" CROWN
1" CRUSHED STONE - PENETRATION METHOD EMULSIFIED ASPHALT SURFACE, SEAL COATED WITH EMULSIFIED ASPHALT AND SAND COVER. STONE = PEA STONE
2" CRUSHED STONE - PENETRATION METHOD EMULSIFIED ASPHALT BASE. STONE GRADED 1 1/2" TO 1 1/2"
6" BANK RUN GRAVEL BASE
INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 1.5 MILES EAST FROM BUSINESS CENTER OF CONCORD, N.H.
NATURAL BASE - COURSE SAND.
6" GRAVEL BASE - BANK RUN GRAVEL LAID IN 2" LAYERS, EACH LAYER WET AND ROLLED WITH TEN TON ROLLER.
2" CRUSHED STONE EMULSIFIED ASPHALT BASE - ASPHALT APPLIED PENETRATION METHOD, 1 GAL. PER SQ. YD. ROLLED, BEFORE & AFTER ASPHALT APPLIED, WITH FIVE TON ROLLER.
1" PEA STONE EMULSIFIED ASPHALT SURFACE - ASPHALT APPLIED PENETRATION METHOD, 1 1/2 GAL. PER SQ. YD. SAND COVER. ROLLED, BEFORE & AFTER ASPHALT APPLIED, WITH FIVE TON ROLLER.
COST OF RUNWAY SURFACE
BASE AND SUB-BASE @ 0.75 PER SQ. YD. EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
G.A.F.
E.E.B
(TITLE) CITY ENGINEER

* W. A. MACATEE
THE ASPHALT INSTITUTE

FIGURE-20

sections of the country. The type used in paving the runways at Rochester is asphaltic macadam, compressed to a final thickness of 2″, on an 8″ rolled stone base in which—it is interesting to note on the drawing—40 per cent run-of-bank sand was used as the filler prior to rolling.

The type of construction used at Rochester also reveals the trend toward somewhat softer grades of asphalt than are normally used in highway construction. The binder was 100/120 penetration asphalt, whereas 85/100 penetration asphalt is frequently used in highway work for constructing asphaltic macadam. Another excellent practice was followed in Rochester whereby the wearing course was seal-coated and enriched with ½ gallon of rapid-curing cutback type of asphalt, per square yard.

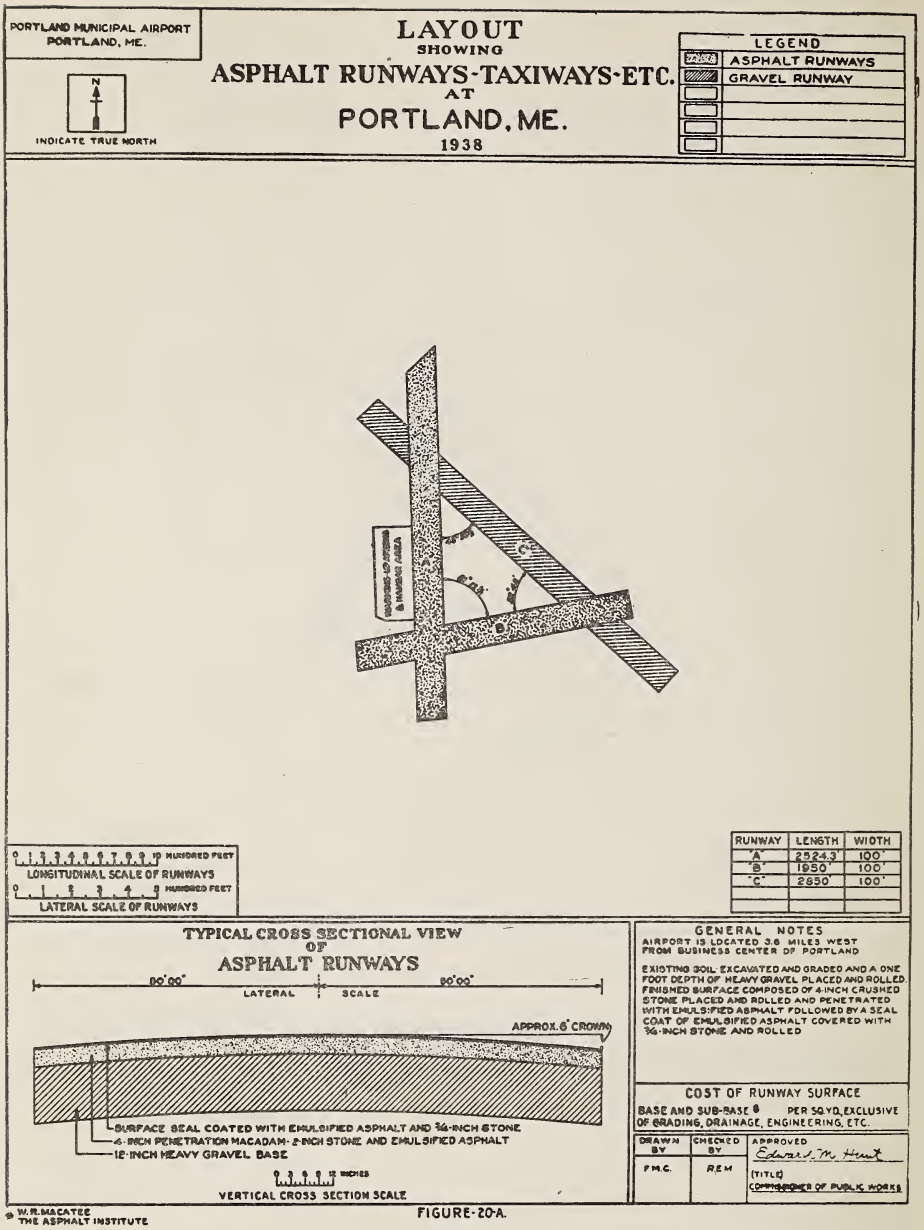


FIGURE-20-A.

DRAWINGS REVEAL CONSTRUCTION METHODS AND DETAILS

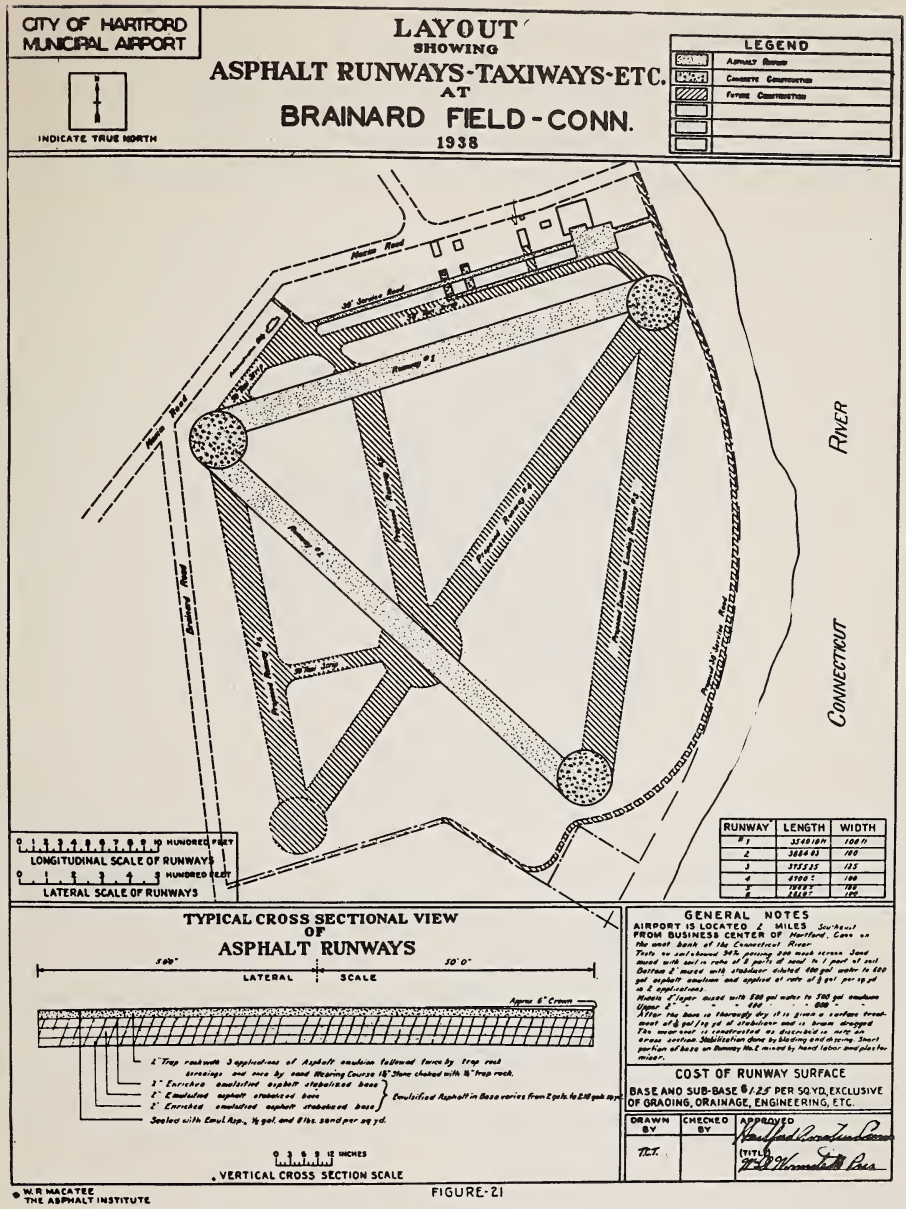
The drawings shown in stereopticon slides and printed form reveal more completely and interestingly the details of construction, dimensions, materials and other features than I could enumerate to you in this general description.

SUMMARY OF TRENDS

The trends in construction of airports as brought out in the replies from many sources are summarized below. The most pronounced trend is listed first, as follows:

1. Toward lighter, less expensive forms of surfacing.

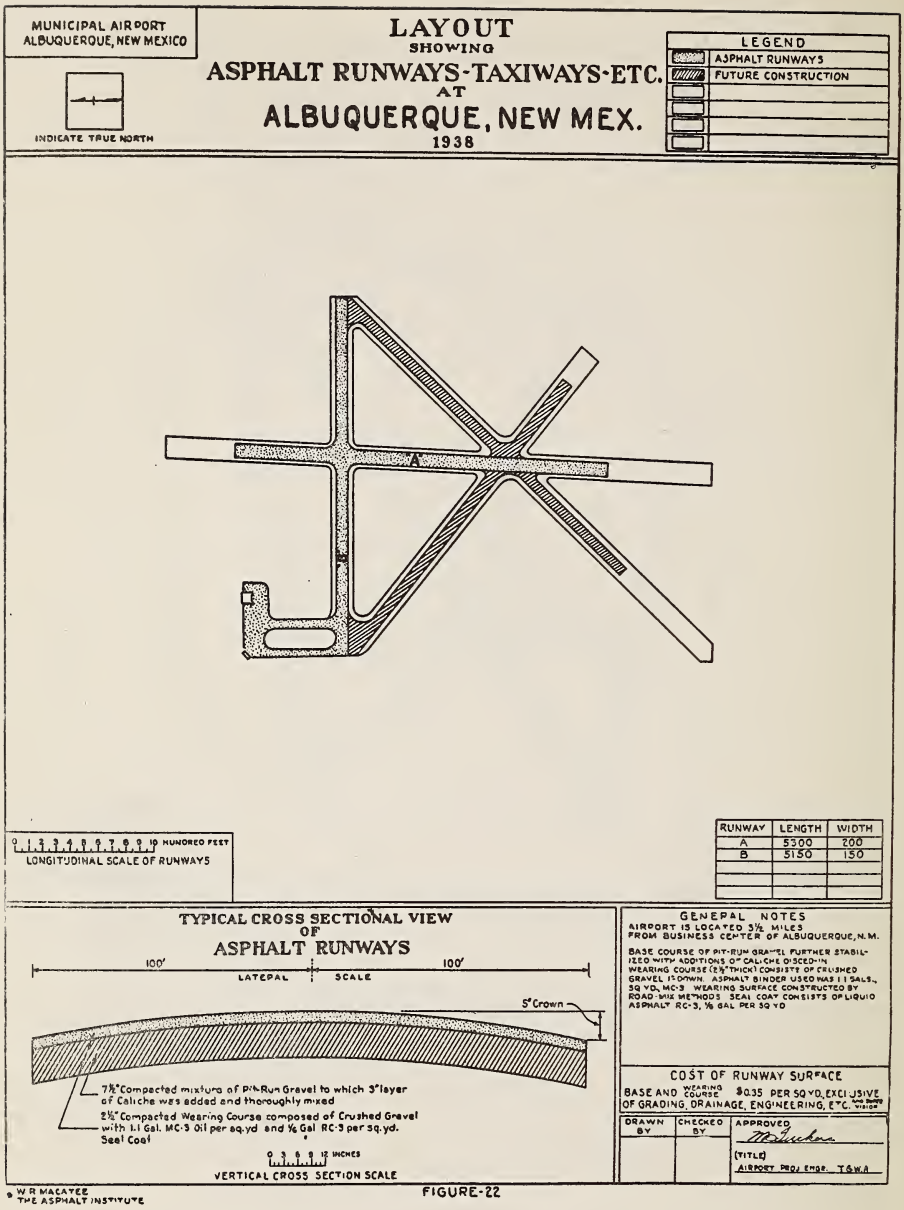
2. Toward utilizing local materials more extensively.


CITY OF HARTFORD
MUNICIPAL AIRPORT
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
BRAINARD FIELD - CONN.
1938
LEGEND
Asphalt Surface
Concrete Construction
Future Construction
Runway #1
Runway #2
Runway #3
Runway #4
Runway #5
Runway #6
Brainard Road
Marsh Road
Airport Road
Proposed Service Road
Proposed Extension Runway #3
Proposed Extension Runway #4
Proposed Extension Runway #5
Proposed Extension Runway #6
Proposed Marsh Road
RIVER
CONNECTICUT
RUNWAY
LENGTH
WIDTH
#1
3540 ft
100 ft
2
3884.43
100
3
3933.35
125
4
4900 ±
100
5
4900 ±
150
6
3680 ±
100
HUNDRED FEET
LONGITUDINAL SCALE OF RUNWAYS
HUNDRED FEET
LATERAL SCALE OF RUNWAYS
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
50'0"
LATERAL SCALE
Approx 6" Crown
1" Trap rock with 3 applications of Asphalt emulsion followed twice by trap rock screenings and once by sand Wearing Course
12" Stone choked with ¾" trap rock.
2" Enriched emulsified asphalt stabilized base
2" Emulsified asphalt stabilized base
2" Enriched emulsified asphalt stabilized base
Emulsified Asphalt in Base varies from 2 gals. to 2.50 gals. per yd.
Sealed with Emul. Asp., ½ gal. and 8 lbs. sand per sq yd.
INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 4 MILES Southeast FROM BUSINESS CENTER OF Hartford, Conn. on the east bank of the Connecticut River.
Tests on soil showed 34% passing 200 mesh screen. Sand mixed with soil in ratio of 3 parts of sand to 1 part of soil. Bottom 2" mixed with stabilizer diluted 400 gal water to 600 gal asphalt emulsion and applied at rate of ½ gal per sq yd in 2 applications.
Middle 4" layer mixed with 500 gal water to 700 gal emulsion
Upper 2" " " 250 " " 300 "
After the base is thoroughly dry it is given a surface treatment of ½ gal/sq yd of stabilizer and is broom dragged. The wearing coat is constructed as described in early cross section. Stabilization done by blasting and mixing. Short portion of base on Runway No. 2 mixed by hand labor and plastic mixer.
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $1.25 PER SQ. YD. EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
TLT
(TITLE)
W. R. MACATEE
THE ASPHALT INSTITUTE
FIGURE-21

3. Toward more extensive use of flexible forms of surfaces and bases.
4. Toward thicker film thicknesses of asphalt surrounding aggregate particles.
5. Toward the use of softer asphalts.
6. Toward greater use of liquid asphalts, such as MC and RC groups.
7. Toward more extensive use of asphalt stabilization of bases.
8. Toward constructing cut-off walls of asphalt mixtures at ends and sides of runways.
9. Toward having on hand fully developed and quickly available plans for future extensions of runways and other airport facilities.

CONCLUSION

I take it that most of you present at this Conference are directly or

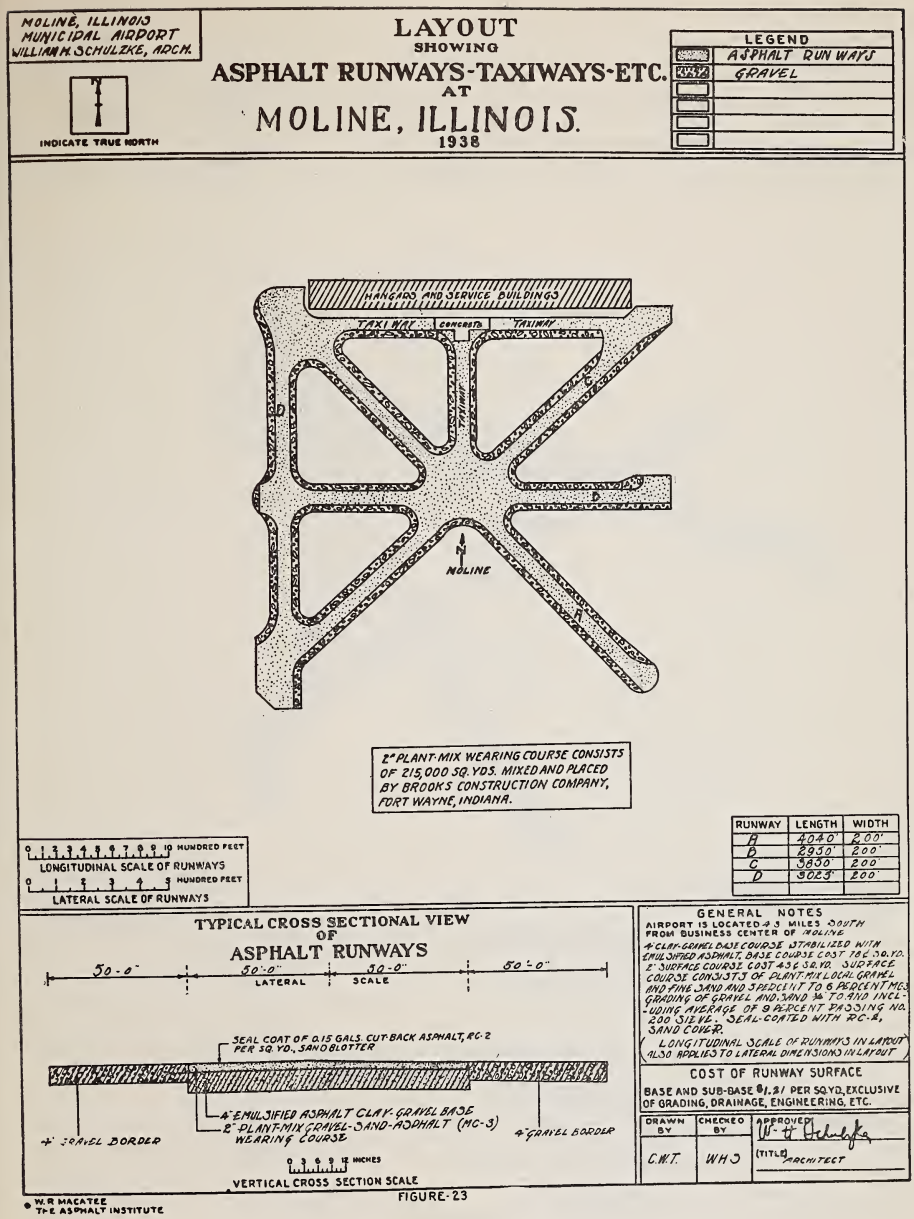

MUNICIPAL AIRPORT
ALBUQUERQUE, NEW MEXICO
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
ALBUQUERQUE, NEW MEX.
1938
LEGEND
ASPHALT RUNWAYS
FUTURE CONSTRUCTION
0 1 2 3 4 5 6 7 8 9 10 HUNDRED FEET
LONGITUDINAL SCALE OF RUNWAYS
RUNWAY | LENGTH | WIDTH
A | 5300 | 200
B | 5150 | 150
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
100'
LATERAL SCALE
100'
5° Crown
7½" Compacted mixture of Pit-Run Gravel to which 3" layer of Caliche was added and thoroughly mixed
2½" Compacted Wearing Course composed of Crushed Gravel with 1.1 Gal. MC-3 Oil per sq. yd and ½ Gal RC-3 per sq. yd. Seal Coat
0 3 6 9 12 INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 5½ MILES FROM BUSINESS CENTER OF ALBUQUERQUE, N.M.
BASE COURSE OF PIT-RUN GRAVEL FURTHER STABILIZED WITH ADDITIONS OF CALICHE DISCED IN
WEARING COURSE (2½" THICK) CONSISTS OF CRUSHED GRAVEL TOP DOWN. ASPHALT BINDER USED WAS 1.1 SALS. SQ YD. MC-3 WEARING SURFACE CONSTRUCTED BY ROAD-MIX METHODS SEAL COAT CONSISTS OF LIQUID ASPHALT RC-3, ½ GAL PER SQ YD
COST OF RUNWAY SURFACE
BASE AND WEARING COURSE $0.35 PER SQ YD, EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
(TITLE)
AIRPORT PROJ. ENGR. T.S.W.A

* W. R. MACATEE
THE ASPHALT INSTITUTE

FIGURE-22

indirectly interested in asphalt types of construction, and therefore zealously endeavor to gain for these types well-earned approbation of your associates and the public. From the large number of replies which I received from officials engaged in constructing and operating airports, I was gratified at the spontaneous, whole-hearted and enthusiastic endorsement which was given asphalt mixtures for surfacing at airports. Sharing with you the same degree of pride in the creditable manner in which asphalt materials serve for surfacing highways and airports, I am glad to quote a sample of the enthusiastic endorsement of these materials which voluntarily came to me from an important engineering official of the United States Navy, reading as follows:

> "Our experience with asphalt-oil treated landing mats and runways has been very satisfactory and we strongly recommend it for such types of soils as occur at this Station."


MOLINE, ILLINOIS
MUNICIPAL AIRPORT
WILLIAM H. SCHULZKE, ADCM.
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
MOLINE, ILLINOIS.
1938
LEGEND
ASPHALT RUN WAYS
GRAVEL
HANGARS AND SERVICE BUILDINGS
TAXI WAY
CONCRETE
TAXIWAY
MOLINE
2" PLANT-MIX WEARING COURSE CONSISTS OF 215,000 SQ. YDS. MIXED AND PLACED BY BROOKS CONSTRUCTION COMPANY, FORT WAYNE, INDIANA.
RUNWAY | LENGTH | WIDTH
A | 4040' | 200'
B | 2930' | 200'
C | 3850 | 200'
D | 3023' | 200'
HUNDRED FEET
LONGITUDINAL SCALE OF RUNWAYS
HUNDRED FEET
LATERAL SCALE OF RUNWAYS
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
50'-0"
50'-0"
LATERAL
50'-0"
SCALE
50'-0"
SEAL COAT OF 0.15 GALS. CUT-BACK ASPHALT, RC-2 PER SQ. YD., SAND BLOTTER
4" EMULSIFIED ASPHALT CLAY-GRAVEL BASE
2" PLANT-MIX GRAVEL-SAND-ASPHALT (MC-3) WEARING COURSE
4" GRAVEL BORDER
4" GRAVEL BORDER
INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 4.3 MILES SOUTH FROM BUSINESS CENTER OF MOLINE.
4" CLAY-GRAVEL BASE COURSE STABILIZED WITH EMULSIFIED ASPHALT. BASE COURSE COST 18¢ SQ. YD.
2" SURFACE COURSE COST 43¢ SQ. YD. SURFACE COURSE CONSISTS OF PLANT-MIX LOCAL GRAVEL AND FINE SAND AND 5 PERCENT TO 6 PERCENT MIS GRADING OF GRAVEL AND SAND 3/4" TO AND INCLUDING AVERAGE OF 9 PERCENT PASSING NO. 200 SIEVE. SEAL-COATED WITH RC-2, SAND COVER.
LONGITUDINAL SCALE OF RUNWAYS IN LAYOUT ALSO APPLIES TO LATERAL DIMENSIONS IN LAYOUT
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $1.21 PER SQ. YD. EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
C.W.T.
W.H.S.
(TITLE) ARCHITECT
W. R. MACATEE
THE ASPHALT INSTITUTE
FIGURE-23

NOTE

Since the foregoing 10 pages were written, based largely on descriptive drawings and other data concerning 18 American airports, there has been received descriptive matter concerning 12 additional airports. It was impracticable to incorporate into the foregoing paper comments with reference to these 12 airports. It seems desirable, however, to now comment upon them and reproduce descriptive drawings concerning them. Therefore, the following comments did not form a part of the foregoing paper but the projects referred to are worthy of description and would have been included in the paper if data concerning them had been received earlier.

Quarry Wastes Utilized

Descriptive drawing showing cross-sectional view of asphalt run-

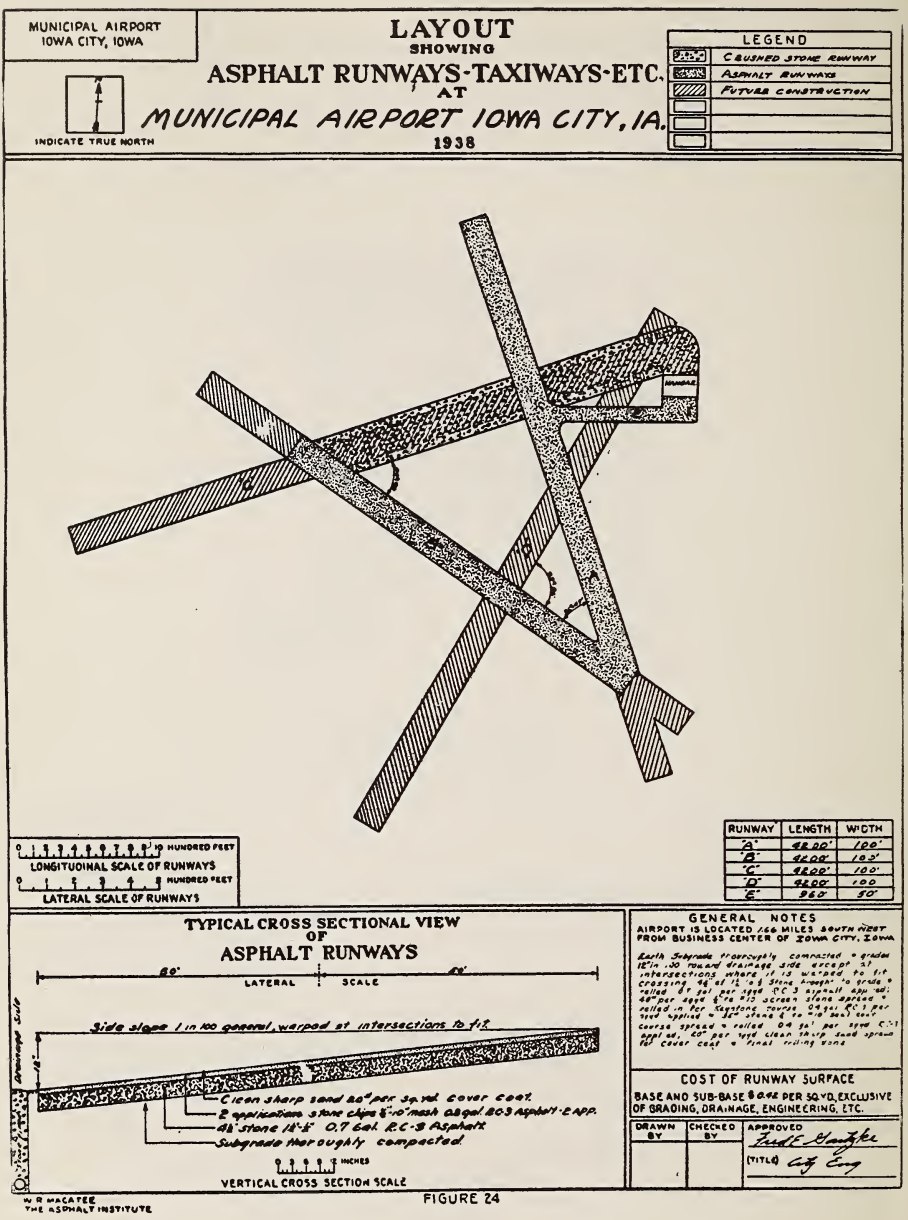


FIGURE 24

ways installed at San Francisco, California (Fig. 18) with the accompanying General Notes, illustrates the desirability of utilizing low-cost aggregates in runways and other asphalt surfaces. In this case, formerly discarded quarry wastes have been used with asphalt to form the base courses at this important Pacific Coast airport.

The same type of aggregates was likewise employed in forming the wearing surface on the runways constructed at Morgantown, West Virginia (Fig. 16). It will be seen that the engineers at Morgantown used liquid asphalt, RC-2, to bind the quarry wastes. Somewhat the same type of mixture, wherein quarry wastes were economically used, was recently carried out by the Kentucky Highway engineers. In the latter case, a kerosene type of cutback asphalt (MC-3) was the binding agent.

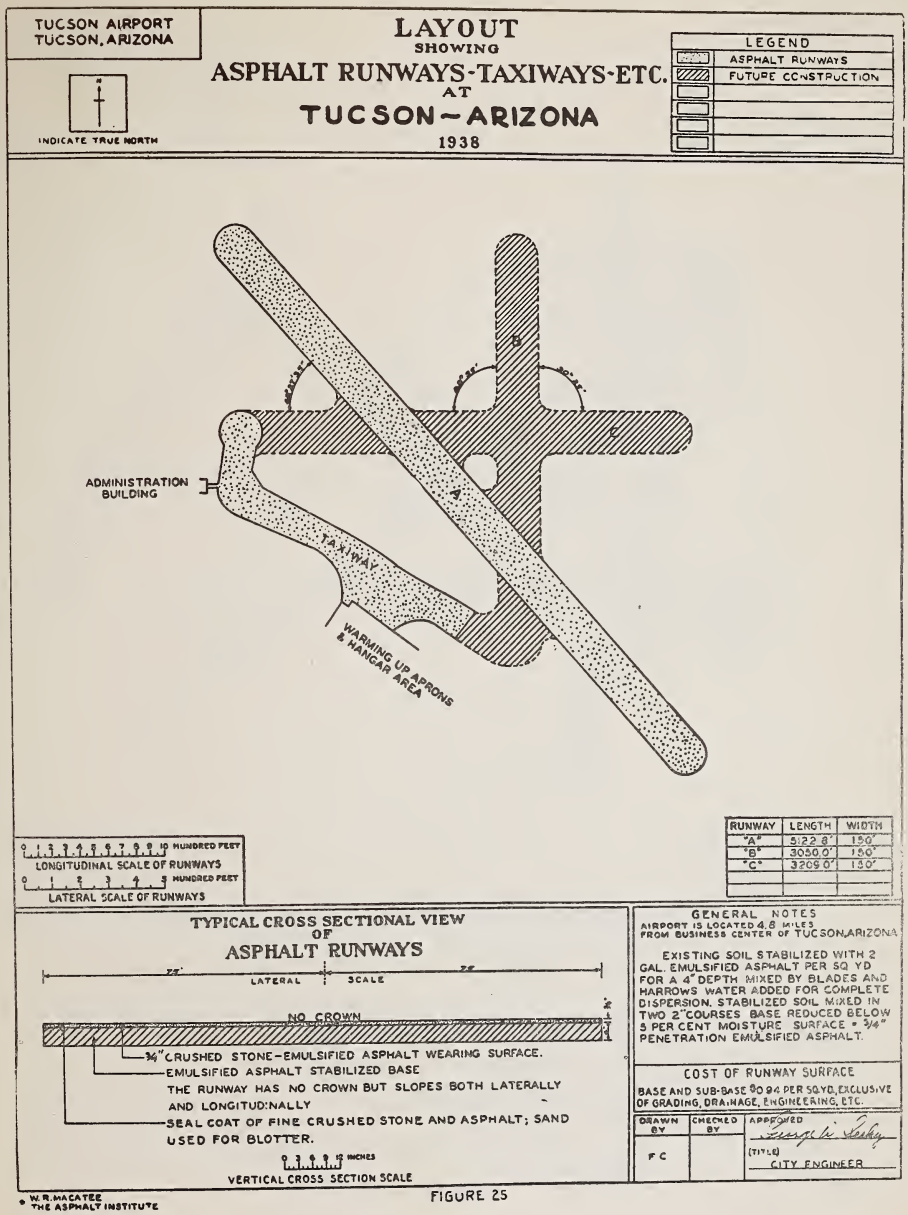

TUCSON AIRPORT
TUCSON, ARIZONA
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
TUCSON—ARIZONA
1938
LEGEND
ASPHALT RUNWAYS
FUTURE CONSTRUCTION
ADMINISTRATION BUILDING
TAXIWAY
WARMING UP APRONS & HANGAR AREA
RUNWAY | LENGTH | WIDTH
"A" 5122.8' 150'
"B" 3050.0' 150'
"C" 3209.0' 150'
HUNDRED FEET
LONGITUDINAL SCALE OF RUNWAYS
HUNDRED FEET
LATERAL SCALE OF RUNWAYS
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
LATERAL SCALE
NO CROWN
¾" CRUSHED STONE—EMULSIFIED ASPHALT WEARING SURFACE.
EMULSIFIED ASPHALT STABILIZED BASE
THE RUNWAY HAS NO CROWN BUT SLOPES BOTH LATERALLY AND LONGITUDINALLY
SEAL COAT OF FINE CRUSHED STONE AND ASPHALT; SAND USED FOR BLOTTER.
INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 4.8 MILES FROM BUSINESS CENTER OF TUCSON, ARIZONA
EXISTING SOIL STABILIZED WITH 2 GAL. EMULSIFIED ASPHALT PER SQ YD FOR A 4" DEPTH MIXED BY BLADES AND HARROWS WATER ADDED FOR COMPLETE DISPERSION. STABILIZED SOIL MIXED IN TWO 2" COURSES BASE REDUCED BELOW 5 PER CENT MOISTURE SURFACE + ¾" PENETRATION EMULSIFIED ASPHALT.
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $0.94 PER SQ YD, EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
F C
(TITLE)
CITY ENGINEER
W. R. MACATEE
THE ASPHALT INSTITUTE

FIGURE 25

MECHANICAL STABILIZATION USED IN CONJUNCTION WITH ASPHALT RUNWAY SURFACES

Where it is more economical to do so, and adequate stability can be secured, it is good practice to employ mechanical stabilization of bases rather than by means of asphalt or other stabilizing agents such as Portland cement or deliquescent salts. Frequently a combination of mechanical and bituminous procedure is most economical.

From a study of the descriptive drawing of the Sioux City, Iowa Airport (Fig. 19), it will be seen that mechanical stabilization was carried out by combining rather large gravel, pea gravel and sand mixed with natural soil. The stabilized base was primed with two-thirds of a gallon of liquid asphalt, MC-1, prior to constructing the asphaltic concrete surfacing of local gravel and 6 per cent of MC-2.

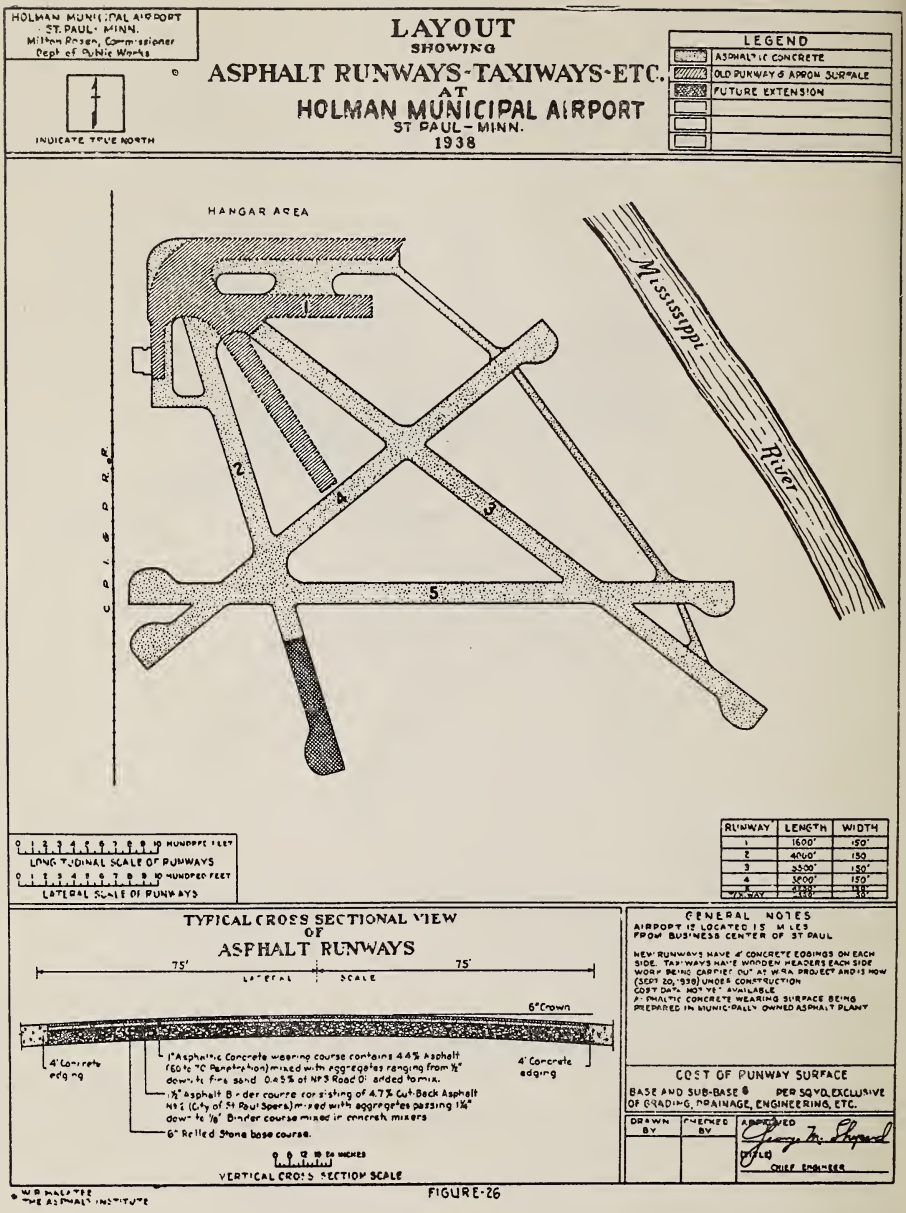

HOLMAN MUNICIPAL AIRPORT
ST. PAUL - MINN.
Milton Rosen, Commissioner
Dept of Public Works
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
HOLMAN MUNICIPAL AIRPORT
ST PAUL - MINN.
1938
LEGEND
ASPHALTIC CONCRETE
OLD RUNWAYS & APRON SURFACE
FUTURE EXTENSION
HANGAR AREA
C. B. & Q. R. R.
Mississippi River
1
2
3
4
5
RUNWAY | LENGTH | WIDTH
1 | 1600' | 150'
2 | 4000' | 150
3 | 3500' | 150'
4 | 3000' | 150'
5 | 4500' | 150
W/S WAY | 4500 | 50
LONGITUDINAL SCALE OF RUNWAYS
LATERAL SCALE OF RUNWAYS
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
75'
LATERAL SCALE
75'
6" Crown
4" Concrete edging
4" Concrete edging
1" Asphaltic Concrete wearing course contains 4.4% Asphalt (60 to 70 Penetration) mixed with aggregates ranging from ½" down to Fine sand 0.45% of No 3 Road Oil added to mix.
1½" Asphalt Binder course consisting of 4.7% Cut-Back Asphalt No 5 (City of St Paul Specs) mixed with aggregates passing 1¼" down to ⅛" Binder course mixed in concrete mixers
6" Rolled Stone base course.
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 15 MILES FROM BUSINESS CENTER OF ST PAUL
NEW RUNWAYS HAVE 4" CONCRETE EDGINGS ON EACH SIDE. TAXIWAYS HAVE WOODEN HEADERS EACH SIDE WORK BEING CARRIED OUT AS WPA PROJECT AND IS NOW (SEPT 20, 1938) UNDER CONSTRUCTION
COST DATA NOT YET AVAILABLE
A - PHALTIC CONCRETE WEARING SURFACE BEING PREPARED IN MUNICIPALLY OWNED ASPHALT PLANT
COST OF RUNWAY SURFACE
BASE AND SUB-BASE @ PER SQ YD. EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
CHIEF ENGINEER
W. P. HALFYER
THE ASPHALT INSTITUTE

FIGURE-26

At Concord, New Hampshire (Fig. 20), the base was made stable by constructing bank-run gravel placed in three 2″ layers prior to placing intermediate course of 2″ asphalt penetration. The final course, or wearing surface, consisted of 1″ asphalt penetration with seal coat in which sand was used as the blotting material.

Another New England example of mechanical base stabilization is illustrated in Fig. 20-A showing the Portland, Maine Municipal Airport. The fact that a 12″ gravel base course was used at Portland, whereas only a 6″ base was used at Concord, New Hampshire, may be accounted for by possible difference in subgrade conditions. As noted earlier in this paper, subgrade-surface thickness relationship governs these matters whereby no uniform thickness of bases can be prescribed, as the strength of subgrades in different localities vary so sharply. It is also to be re-

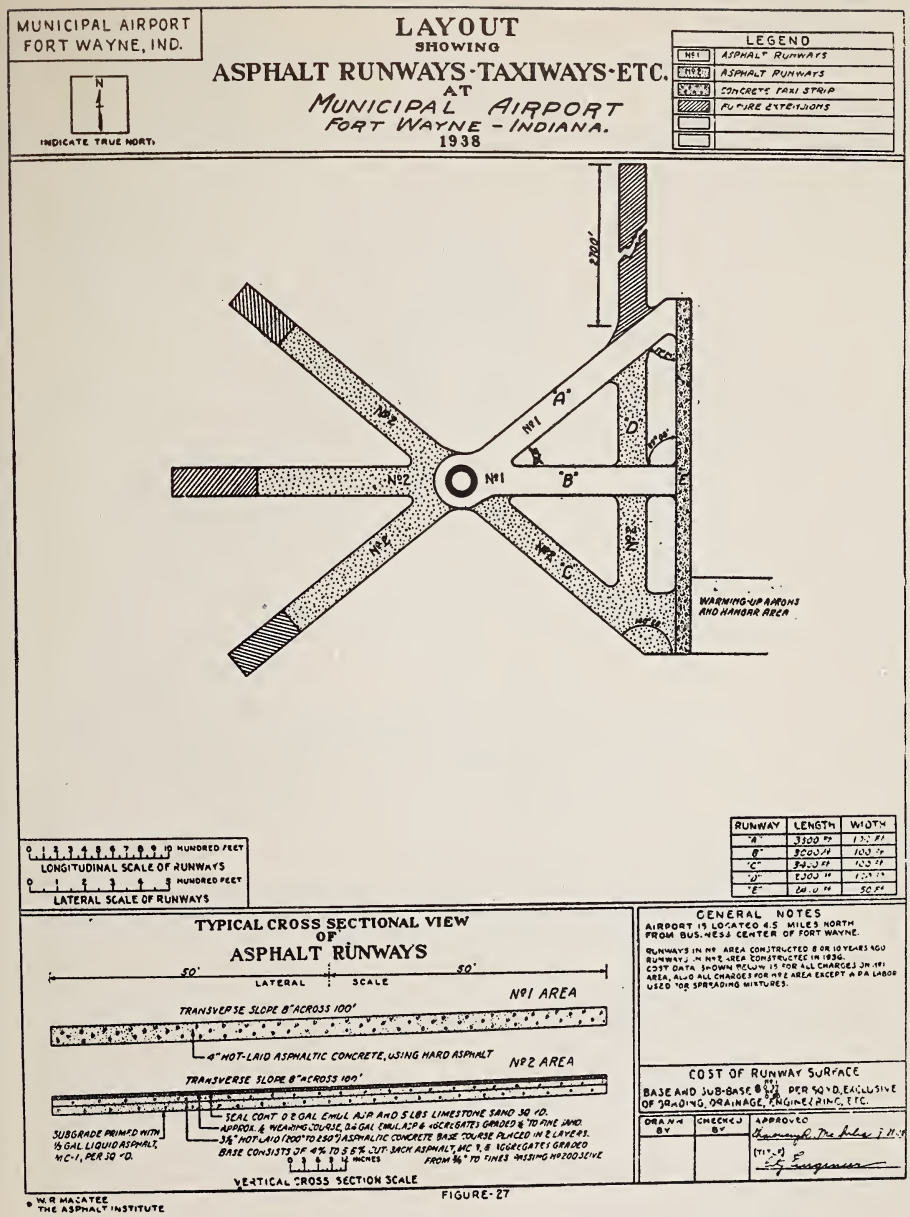

MUNICIPAL AIRPORT
FORT WAYNE, IND.
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
MUNICIPAL AIRPORT
FORT WAYNE - INDIANA.
1938
INDICATE TRUE NORTH
LEGEND
ASPHALT RUNWAYS
ASPHALT RUNWAYS
CONCRETE TAXI STRIP
FUTURE EXTENSIONS
2200'
A
B
C
D
E
No1
No2
WARMING-UP APRONS AND HANGAR AREA
RUNWAY | LENGTH | WIDTH
"A" 3300 FT 150 FT
"B" 3000 FT 100 FT
"C" 3400 FT 100 FT
"D" 2300 FT 150 FT
"E" 2400 FT 50 FT
LONGITUDINAL SCALE OF RUNWAYS
LATERAL SCALE OF RUNWAYS
HUNDRED FEET
TYPICAL CROSS SECTIONAL VIEW OF ASPHALT RUNWAYS
50' LATERAL SCALE 50'
No1 AREA
TRANSVERSE SLOPE 8" ACROSS 100'
4" HOT-LAID ASPHALTIC CONCRETE, USING HARD ASPHALT
No2 AREA
TRANSVERSE SLOPE 8" ACROSS 100'
SEAL COAT OF GAL EMUL ASP AND 5 LBS LIMESTONE SAND PER SQ YD.
APPROX. 1" WEARING COURSE, 2 GAL EMUL ASP & 10 LBS AGGREGATES GRADED TO FINE SAND
3" HOT-LAID (200° TO 250°) ASPHALTIC CONCRETE BASE COURSE PLACED IN 2 LAYERS.
BASE CONSISTS OF 4% TO 5.5% CUT BACK ASPHALT, MC 3, & 100 LBS AGGREGATES GRADED FROM 1/2" TO FINES PASSING NO 200 SIEVE
SUBGRADE PRIMED WITH 1/2 GAL LIQUID ASPHALT, MC-1, PER SQ YD.
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 4.5 MILES NORTH FROM BUSINESS CENTER OF FORT WAYNE.
RUNWAYS IN No1 AREA CONSTRUCTED 8 OR 10 YEARS AGO
RUNWAYS IN No2 AREA CONSTRUCTED IN 1938.
COST DATA SHOWN BELOW IS FOR ALL CHARGES IN No1 AREA, ALSO ALL CHARGES FOR No2 AREA EXCEPT WPA LABOR USED FOR SPREADING MIXTURES.
COST OF RUNWAY SURFACE
BASE AND SUB-BASE $ PER SQ YD, EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY
CHECKED BY
APPROVED
(TITLE)

FIGURE-27

* W. R. MACATEE
THE ASPHALT INSTITUTE

membered that gravel is very cheap in New England, so that critical thickness in respect to cost is not so important.

The airport at Albuquerque, New Mexico (Fig. 22), is another example of using a combination of locally available soils to economically construct a mechanical type of base stabilization. It will be observed that the engineers utilized pit-run gravel and caliche.

COMBINED ASPHALT STABILIZATION AND MECHANICAL STABILIZATION

The bases used in the runways constructed at Hartford, Connecticut (Fig. 21), are combinations of asphalt stabilization and mechanical stabilization. It will be noted from descriptive drawing that the soil was partly stabilized by addition of sand. This procedure did not give desired re-

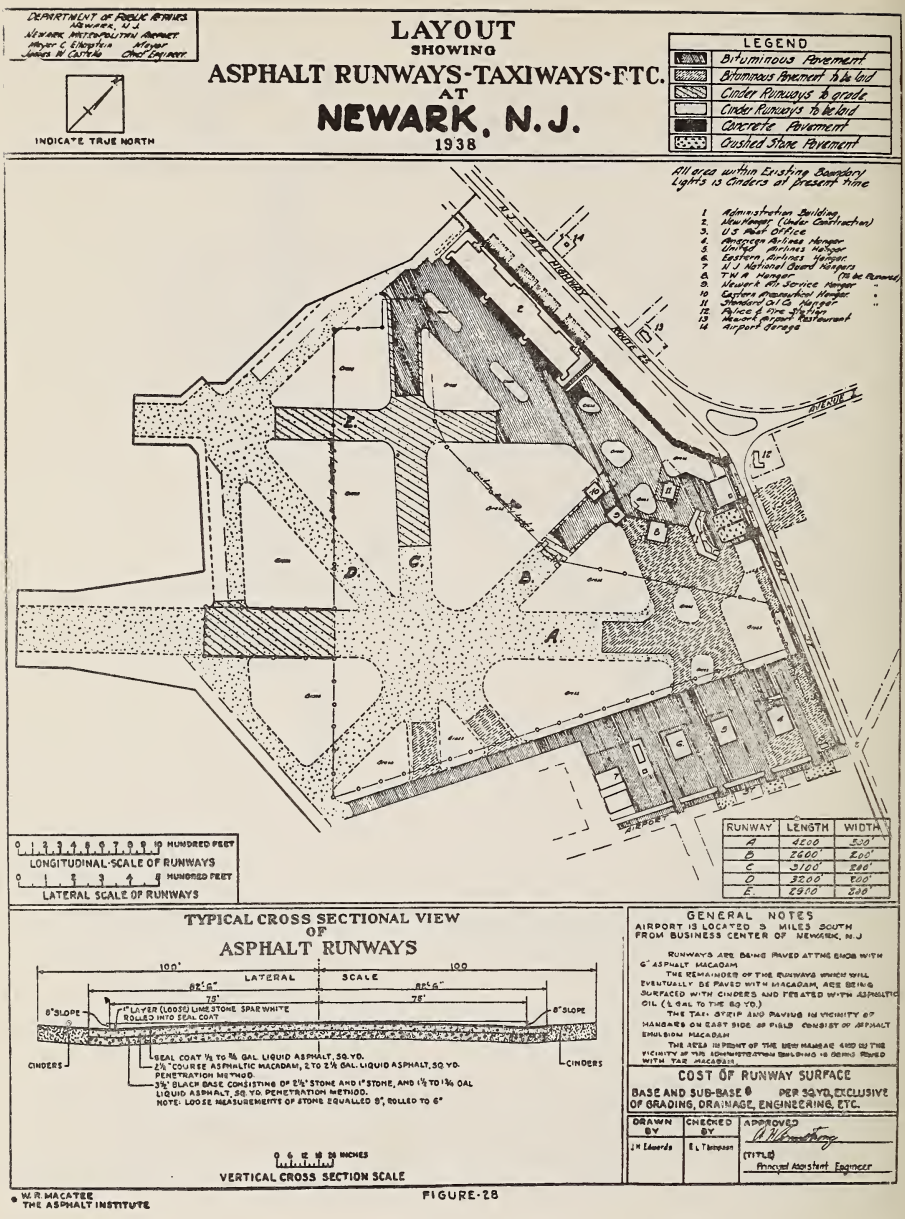

DEPARTMENT OF PUBLIC AFFAIRS
NEWARK, N.J.
NEWARK METROPOLITAN AIRPORT
Mayor C. Elmo Tolin
Mayor
Commr. W. Curtis
Chief Engineer
INDICATE TRUE NORTH
LAYOUT
SHOWING
ASPHALT RUNWAYS-TAXIWAYS-ETC.
AT
NEWARK, N.J.
1938
LEGEND
Bituminous Pavement
Bituminous Pavement To be laid
Cinder Runways To grade
Cinder Runways To be laid
Concrete Pavement
Crushed Stone Pavement
All area within Existing Boundary Lights is Cinders at present time
1 Administration Building
2 New Hangar (Under Construction)
3 U.S. Mail Office
4 American Airlines Hangar
5 United Airlines Hangar
6 Eastern Airlines Hangar
7 N.J. National Guard Hangars
8 T.W.A. Hangar (To be Removed)
9 Newark 4th Service Hangar "
10 Eastern Transactions Hangar "
11 Standard Oil Co. Hangar "
12 Police & Fire Station
13 Newark Airport Restaurant
14 Airport Garage
N.J. STATE HIGHWAY
ROUTE 25
AVENUE A
FORT
RUNWAY | LENGTH | WIDTH
A | 4500' | 300'
B | 2600' | 200'
C | 3100' | 200'
D | 3500' | 200'
E | 2900' | 200'
HUNDRED FEET
LONGITUDINAL-SCALE OF RUNWAYS
HUNDRED FEET
LATERAL SCALE OF RUNWAYS
TYPICAL CROSS SECTIONAL VIEW
OF
ASPHALT RUNWAYS
100'
LATERAL SCALE
100
82'-6"
78'
6° SLOPE
1" LAYER (LOOSE) LIME STONE SPAR WHITE ROLLED INTO SEAL COAT
SEAL COAT 1/2 TO 3/4 GAL. LIQUID ASPHALT, SQ. YD.
2 1/2" COURSE ASPHALTIC MACADAM, 2 TO 2 1/2 GAL. LIQUID ASPHALT, SQ. YD. PENETRATION METHOD
3 3/4" BLACK BASE CONSISTING OF 2 1/2" STONE AND 1" STONE, AND 1 1/2 TO 1 3/4 GAL LIQUID ASPHALT, SQ. YD. PENETRATION METHOD.
NOTE: LOOSE MEASUREMENTS OF STONE EQUALLED 8", ROLLED TO 6"
CINDERS
0 6 12 18 24 INCHES
VERTICAL CROSS SECTION SCALE
GENERAL NOTES
AIRPORT IS LOCATED 5 MILES SOUTH FROM BUSINESS CENTER OF NEWARK, N.J.
RUNWAYS ARE BEING PAVED AT THE ENDS WITH 6" ASPHALT MACADAM.
THE REMAINDER OF THE RUNWAYS WHICH WILL EVENTUALLY BE PAVED WITH MACADAM, ARE BEING SURFACED WITH CINDERS AND TREATED WITH ASPHALTIC OIL (1 GAL TO THE SQ YD.)
THE TAXI STRIP AND PAVING IN VICINITY OF HANGARS ON EAST SIDE OF FIELD CONSIST OF ASPHALT EMULSION MACADAM.
THE AREA IN FRONT OF THE NEW HANGAR AND IN THE VICINITY OF THE ADMINISTRATION BUILDING IS BEING PAVED WITH TAR MACADAM.
COST OF RUNWAY SURFACE
BASE AND SUB-BASE @ PER SQ YD, EXCLUSIVE OF GRADING, DRAINAGE, ENGINEERING, ETC.
DRAWN BY J.H. Edwards
CHECKED BY E.L. Thompson
APPROVED
(TITLE) Principal Assistant Engineer

FIGURE-28

• W.R. MACATEE
THE ASPHALT INSTITUTE

sults. Therefore, the engineers combined mechanical stabilization with asphalt stabilization. This is always good practice if the economic factors revealed in a study of the situation indicate the need for and the possibility of using this dual form of stabilization. Aside from developing the desired stability, this type gives assurance that the integrity of the base will be preserved.

LOOSE AGGREGATE BORDERS

The airport at Moline, Illinois (Fig. 23) immediately across the river from Burlington, Iowa, has an unusual form of borders whereby loose gravel 4″ thick and 50′ wide on each side augment the 100-foot asphalt runways constructed with 6 per cent of liquid asphalt, MC-3, and seal-coated with 0.15 gallons of cutback asphalt, RC-2, per square yard. This surfacing

s now in the process of being placed and is expected to be completed he latter part of October, 1938.

This plant-mixed type of surfacing is being mixed, placed and rolled it the rate of 600 to 800 tons a day of 20 hours each, beginning at 2 A. M. and ending at 10 P. M. An interesting feature of this construction is the fact that portable, mobile flood lights are used to advantage n night operations. The lights not only permit night work being carried on, but clearly reveal surface inequalities, permitting corrections to be nade as work progresses.

UPPER PORTION OF BASE STABILIZED

The Municipal Airport at Iowa City, Iowa (Fig. 24) employs a form of base construction which is a modification of asphalt stabilization wherein a composite form of stabilization and prime coating is carried out by penetrating the upper portion of the base with 0.7 gallons of cutback asphalt, RC-3, per square yard. This 4½″ stone base course serves to support a wearing surface composed of stone chips and RC-3 liquid asphalt applied in two applications. Here again it will be noted that clean, sharp sand is much favored as a cover coat for sealing asphalt surfaces.

Another example of coarse sand blotter is shown in the Tucson, Arizona Airport (Fig. 25) which brings out quite clearly the pronounced trend toward use of fine-grained particles for seal-coating purposes. This form of construction clearly increases skid-resistance.

COMBINED USE OF LIQUID AND HARD ASPHALTS

Holman Field, the municipal airport at St. Paul, Minnesota (Fig. 26) reveals an example of combined use of liquid asphalt, identified as cutback asphalt No. 2 (RC-2) which is used in concrete mixers for producing 1½″ binder course on which is being placed a 1″ wearing surface of asphaltic concrete in which 60/70 penetration asphalt cement is the binder. The wearing surface on this important Northwest airport is being produced in the municipally-owned asphalt plant, whereas, the binder course was mixed at the airport site in concrete mixers. The chief reason for using these two types of asphalt was to utilize to the greatest possible extent WPA labor, thus increasing the ratio of labor cost to material costs.

LIGHT COLORED SURFACING

Descriptive drawing of the Municipal Airport at Fort Wayne, Indiana (Fig. 27) again reveals a pronounced trend toward less expensive forms of runway surfaces. It will be observed that Area No. 1, constructed 8 or 10 years ago, cost approximately 20 per cent more than the surfacing in Area No. 2 constructed in 1936, exclusive of the item of WPA labor required for spreading. It will be noted that the seal coat used in the 1936-program employed 5 pounds of limestone-sand per square yard. The use of fine-grained, light-colored aggregates for seal coats, such as limestone-sand, is especially advantageous on airports for the reason that it increases surface visibility, especially in night flying.

The big Newark Airport (Fig. 28) which serves the metropolitan areas of that city—in addition to providing the major portion of New York City's air terminal facilities—is another example wherein applications were made of light-colored aggregates to provide highly satisfactory surface visibility.

BLACK BASE

Numerous examples of black base type of airport runway construction using plant-mix methods, were reported—such as the airports at Salt Lake City, Los Angeles Harbor and Charleston, South Carolina. The drawing showing the heavy-duty air field at Newark, which probably serves more incoming and outgoing planes than most any other of the country's airports, illustrates the use of black base wherein penetration method of construction was employed, using asphalt.

STAGE-CONSTRUCTION

The Newark Airport is located on made-land located on what is locally known as "the Jersey meadows." This was a swamp and, therefore, very unstable and required a hydraulic fill of 5,500,000 cubic yards. In addition, 2,000,000 cubic yards of dry fill, or cellar dirt, was hauled in. Due to the unstable nature of the site, construction has proceeded relatively slow, whereby temporary surfaces of oiled cinders are first constructed, and when the subgrade has become reasonably stable, the flexible asphaltic black base and surface course has been constructed. This has required stage-construction for which asphalt is notably adapted.

PERIODICAL INSPECTIONS DESIRABLE

As noted elsewhere in this paper, thick films of asphalt are desirable in constructing asphalt mixtures for airports, without reducing stability below the minimum required to prevent displacement under traffic. It is also indicated that engineers responsible for designing airport surfaces are conscious of the need for employing thick films of asphalt around particles of aggregate. This trend toward using somewhat larger quantities of asphalt than may be necessary in mixtures for highways, should not be regarded as making unnecessary occasional surface treatments to preserve an impermeable surface to prevent oxidation by action of air and water.

It is, therefore, desirable that periodical inspections be made of asphalt surfaces of runways—preferably in spring and fall of the year—to determine whether or not surface treatments are needed. The most accurate method for determining need for surface treating asphalt mixtures is by the use of a test apparatus, known as the Permeometer.*

This device, consisting of a graduated, funnel-like tube holding a given quantity of water, is sealed tightly to cleaned airport surfaces and water poured into it. The rate of leakage is then observed to determine the need for seal-coating the surface.

*(Detailed drawing showing Permeometer apparatus is illustrated on Page No. 510 of 1938 Convention Proceedings, American Road Builders' Association).

PERMEABILITY LIMITS

The following rules concerning the need for sealing roadway surfaces have been prescribed by the California Division of Highways. The same rules seem equally appropriate for airport surfaces:

"Sealing should be required on various surfaces when permeability tests show leakage in excess of 2½ cc. per sq. in. of area tested in less than the period of time reported below:

On bituminous macadam	120 seconds
On oil mixes, cutbacks, etc.	180 seconds
On asphalt concrete	180 seconds

PREVENT PLANT GROWTH

Because of the nature of traffic at airports, it is desirable to render completely sterile the underlying soil to prevent the growth of grass through flexible surfaces. Grass will not grow through flexible forms of surfacing in all regions but certain types of grass in some regions will prove harmful unless the soil is rendered sterile. Sterilization of soil can be inexpensively and completely carried out with the use of chemicals, such as rock salt (sodium chloride) or sodium chlorate. It is applied usually in crystalline form to the surface of the soil, prior to placing first course of pavement. Experts on this subject are usually to be found at sub-stations of the Federal or State Departments of Agriculture.

Drawings shown in Appendix give essential and correct details. In some cases slight changes have been made to more fully reveal methods of construction. In a few cases, drawings were redrawn, based on original plans and other authentic descriptive matter, to come within appropriate layout limits.

An effort was made to include in the foregoing paper descriptive data of airport suitable for small, intermediate and large communities. It was also the writer's purpose to incorporate into it various forms and types of asphalt construction to illustrate (a) simple surface treatments, (b) road-mix types, (c) mixed-in-place construction, using traveling plants, (d) hot-mixed types, (e) cold-laid forms of plant-mix construction, (f) asphalt soil stabilization, (g) sand-asphalt and (h) the highly stable and economical "black base" forms of construction.

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Mississippi's Experience With Loss in Penetration

By

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Prior to 1936, the only funds available for road construction in Mississippi were those obtained from the Government as direct grants for unemployment or county bond issues used to match Federal aid. As a consequence, our State Highway System of nearly six thousand miles was mostly gravel, and the alignment of the horse and buggy vintage.

It was not until 1932, therefore, in the First Emergency Program, that any hot-mix asphalt pavements were laid. At this time contracts were awarded for 20 miles of 4½″ asphaltic concrete pavement consisting of a 3″ asphaltic concrete base course with a 1½″ modified Topeka wearing course. The base course design specified that 50% of the aggregate should be crushed, which necessitated shipping in slag from Birmingham, or crushed gravel, as no crushed limestone or trap-rock is available in Mississippi. For the wearing course 95% of the aggregate retained on the No. 4 mesh sieve was crushed. The apparent specific gravity was about 2.25, making the density (% of theoretical), 92%. 40-50 penetration asphalt was used, the majority of the cars having a penetration of 43-45 @ 77° F. The addition of this long haul, expensive material, therefore, ran the cost of these projects up to $16,000.00 per mile for the paving items.

These pavements were laid on an 8″ compacted sand-clay gravel base, having a P. I. of 6 to 8 with a L. L. of 25. Unfortunately, however, this gravel base was placed, in many sections, on a very active clay subsoil, which classifies as an A-6 material, our Soils Laboratory at that time being in its infancy.

Some two or three years after completion, it was noted that on some sections of these projects excessive cracking was developing. Samples taken indicated this cracking to be confined to the wearing surface only and was not in evidence in the base course. This cracking was in evidence only on those sections of bad soil, but there was no deformation in cross-section.

This condition progressed to where some cracking developed over most of the project, and samples taken when the pavement was five years old showed the cracks had progressed into the base course, and the penetration of the asphalt, by the Abson method, was down to 24,

a loss of 44% in five years. Those sections, however, where the subbase was of good soil, show considerably less cracking, in some instances no cracking at all.

These results check with the results obtained in other States as to loss in penetration. It would appear, however, that a part of this cracking is due to the so called "drying out" of the mix, rather than to the hardening of the asphalt. The conclusion is reached, therefore, that to the quantity of asphalt used, as well as the loss in penetration, can be attributed the cause of this cracking. It is obvious, therefore, that the quantity of asphalt cement used should be the maximum without producing a slippery surface.

It is interesting to note that the surface course mixtures which, in general, contain not less than 6% of the mass passing the No. 200 mesh sieve, show a loss of ten to fifteen per cent less than the more open mixtures, such as are generally used in base courses. We are running frequent checks on the penetration loss from various plants in operation, and find that the average loss in penetration occurs in the mixer box. A sample of the raw asphalt is obtained on the mixer platform as it enters the mixer box, and a sample of the completed mix taken from the truck as the mix is discharged into it. With a mixing time of 45 and 15 seconds, a loss of 12 to 15% occurs on 80 penetration asphalt from mixer box to truck. Samples taken from the completed pavement several days after final rolling show no further loss.

One plant with a rotary type mixer is in operation in Mississippi, and several pug mill plants. Tests have been made to see if the type paving plant affects in any way the loss in penetration. To date we have been unable to detect any difference.

The mixing temperature on all our work is determined by the minimum temperature required to properly dry the aggregate. This temperature does not exceed 325° F. and is more frequently around 250°. Between a mixing temperature of 235° and 350,° the loss in penetration is the same with the same mixing period, which indicates that oxidation due to agitation, rather than high temperature, is the cause of the penetration loss. This is further borne out by one small pug mill plant on which the mixing time was set for 1½ minutes. The loss was 25% in the mixer box, using a 200 penetration asphalt.

Recently a local aggregate deposit has been developed that has a loss by wash of 10 to 15%. Of this material approximately 5% is clay having a low P. I. Extractions made with benzene appear to give erroneous results, as a portion of the asphalt cement is adsorbed by the clay particles to such an extent that the benzene cannot dissolve them. As a consequence, it appears that all of the bitumen cannot be recovered from the mix by the usual methods, and it is believed that, under such conditions, the results obtained are not indicative of the actual penetration of the asphalt in the pavement. Laboratory mixes using this material have been sent to a commercial laboratory, as well as the Arlington Farms Laboratory, for a check on our methods. We are unwilling at this time, however, to venture any conclusions, but our investigations indicate that all of the bitumen must be recovered to obtain accurate results.

On inverted penetrations we have noticed that on some projects the aggregate is gradually whipped off while the surface remains intact. This will eventually result in a slippery section, and gives the surface a rather "mangy" appearance. Due to our use of tar prime on this work, we are unable to determine the penetration on the recovered asphalt, but believe that there is a considerable loss in penetration.

From the work done in our Laboratory and the reports on work done by other States, a voluminous mass of data on the behavior of asphalts is being obtained. These data are sometimes so inter-related and obtained from so many different angles of approach as to be confusing. From this work, however, we believe the behavior of plant mix work in Mississippi will permit one conclusion to be reached. This conclusion is that the use of softer asphalts, while desirable, will not entirely eliminate detrimental cracking, unless the maximum asphalt content permitted by safe practice is used. It is felt that this is particularly true in the South, where cracking of asphaltic types of pavement constructed on an adequate base has resulted as much from a lack of asphalt as from a loss in penetration.

Plantmix Bituminous Surface Construction in Nevada

By

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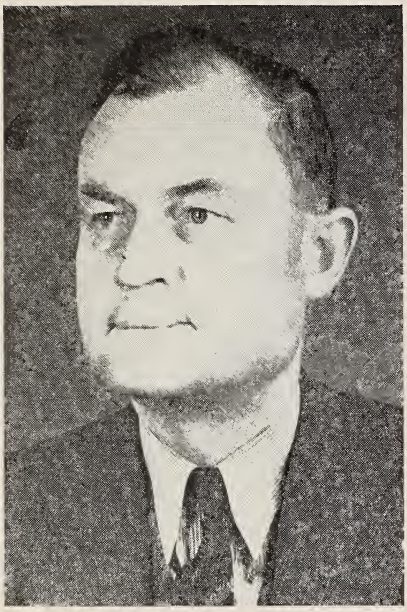

During the year 1932 Nevada placed its first plantmix bituminous surface. While the construction of this type of surface was experimental with us, sufficient information was at hand from other states to lead to the belief that it would prove successful and warrant the additional cost over and above the cost of road-mixes.

This surface was laid on a plantmix crushed rock base, and a brief description of the construction methods employed, together with specifications for both base and surface, follow. A four inch compacted thickness of base and three inch compacted surface were required.

CRUSHED ROCK BASE COURSE

Grading of Mineral Aggregates

Passing 2½ inch screen, circular openings	100%
Passing 3 mesh sieve	30- 50%
Passing 200 mesh sieve	7- 14%

Proportioning and Mixing

The material was separated into two sizes on three mesh screen and each size stored in a separate bin. These two sizes of material were recombined in proper proportion within the limits specified. The properly proportioned aggregates were then mixed with water at a central mixing plant in either a rotary drum or pug mill type mixer. Sufficient water was used to thoroughly dampen the material and prevent segregation. This required between nine and eleven gallons per ton of dry aggregate.

Placing

The material was placed in one course on approved subgrade using Hempbell type spreader boxes in order to secure the required tonnage per station. It was then spread with blades to proper crown and cross section.

Compaction

The material was compacted by rolling with 8-ton tandem rollers.

It was then finished by adding water and rolling until a thin grout flushed along ahead of the roller. Immediately after the surface dried sufficiently, a prime coat of light fuel oil corresponding to SC-1A was applied at the rate of ¼ gallon per square yard of surface. Prior to applying the prime coat, the surface was thoroughly broomed to remove all loose particles.

PLANTMIX BITUMINOUS SURFACE COURSE

Grading of Aggregate

Passing 1 inch screen, circular openings	100%
Passing No. 3 mesh sieve	40- 70%
Passing No. 10 mesh sieve	35- 60%
Passing No. 200 mesh sieve	7- 14%

Proportioning and Mixing

The material was separated into two sizes on three mesh screen and each size stored in separate bin. These two sizes were recombined in proper proportions within the limit specified. All material was proportioned accurately by weight.

The properly proportioned aggregate was passed through a dryer and then mixed with cutback asphalt similar to an MC5 in pug mill type mixer. The amount of cutback asphalt used averaged 4.8% of the weight of the dry aggregate. The mixed material reached the road at a temperature of between 175° and 200° F.

Placing

The material was placed in one course on the approved base, using spreader boxes. It was then spread uniformly to the depth loose which would give the compacted thickness required. Immediately after being spread, the material, except the bottom one to one and one half inches, was bladed into a windrow on one side of the road and then evenly spread over the entire surface. This operation was repeated if found necessary until the entire surface had a uniform texture, smooth, true to cross section and profile, and uniformly compacted.

Rolling

Rolling operations were performed with 8-ton tandem rollers traveling longitudinally and progressing inward from the shoulders.

The plantmix base was constructed to a width of twenty-two feet and the surface to a width of twenty feet.

The surface on this project was not sealed until two years after completion. Unit prices were: Base, $1.32 per ton; Surface, $2.28 per ton.

The price for surface course includes the cost of asphalt. The cost per mile for base, including prime coat, was $3,800 and for surface course, $4,400.

At this writing the surface shows few signs of breaking up, and although there has been some fill settlement even at these points, it retains its original liveness and waterproof texture.

Since this first job, we have experimented with various types of asphalts and made slight changes in aggregate grading requirements, also in method of placing, spreading, and finishing. The principal changes of modifications in the specifications consist of the following:

Aggregates

The mineral aggregates are now required separated into three sizes, each size stored in a separate bin. The bin sizes are:

No. 1 Bin — 1 inch to No. 3
No. 2 Bin — No. 3 to No. 10
No. 3 Bin — Minus No. 10

Combined grading specified.

Passing 1 inch screen, square openings	100%
Passing ½ inch screen	65- 80%
Passing No. 4	45- 60%
Passing No. 10	30- 45%
Passing No. 40	15- 35%
Passing 200 mesh	5- 12%

Bituminous Binder

Lately we have been specifying 90 to 95 asphalt or SC6, but have used SC4, MC3 and 4, and 95 plus with varying results.

Spreading and Finishing

Two mechanical spreading and finishing machines were tried, resulting in the following specification:

> "The paver shall be an approved, self-propelled type which will spread and strike off a width equal to one travel lane (multiples of ten or eleven feet), in such manner that no supplemental shaping will be required. It shall be so designed that neither the traction nor the main load of the paver will be carried on the uncompacted mix and shall be equipped with an agitator, preferably of the pug mill type. The paver shall spread the material evenly and with uniform density and strike it off true to the required cross section. Screeds shall not follow minor irregularities in the base, shall be readily adjustable, and shall be provided with gauges for accurate control of depth of spread."

Rolling is performed with 8-ton tandem rollers in the same manner as previously outlined.

We have not been entirely satisfied with the surface obtained using the mechanical spreaders as on practically every job bumps developed which could not be smoothed out with the tandem rollers. This surface roughness is attributed to the starting and stopping of the machine and also to varying compaction caused each time a load is dumped in the hopper. On one job a three-axle tandem roller was specified in an endeavor to secure a smoother riding surface. The results obtained proved much more satisfactory, and this type of roller will be required on all future jobs where a mechanical finisher is specified.

We have found that a smoother riding surface results from spreading and finishing plantmix material with blades, but of course when the heavier asphalts are specified this method can not be employed.

Present design requires a twenty-two foot width of plantmix bituminous surface of a compacted thickness of two and one-half inches placed on a four-inch compacted thickness of plantmix crushed rock base which is not less than twenty-four feet in width.

On last year's projects, we experimented by placing a bituminous seal on all plantmix surfaces. This consisted of a seal coat of approximately one fifth gallon of penetration type emulsified asphalt covered with fifteen pounds per square yard of screenings graded from one-half inch to No. 10 in size. The screenings were immediately broomed with a drag broom and rolled.

Low bid prices during the past two years for plantmix bituminous surfaces and plantmix crushed rock base range from $1.86 to $3.13 per ton for the former and $0.88 to $1.45 per ton for the latter. The average cost per mile is $3,960 for the base and $4,590 per mile for the surface.

Naturally prices vary according to the magnitude of the job, location of material deposits in relation to the project, ease of production of the aggregate, and proximity of rail facilities for bituminous materials.

Record of Construction of Black Base Asphalt Macadam Penetration Type

OKLAHOMA F. A. P. 22-A (Reop.) COMANCHE COUNTY

By

CARL R. REID,
Engineer of Materials,
Oklahoma State Highway Commission

The project here described included 6.293 miles of Asphalt Macadam Base Course, six (6) inches compacted thickness and 24 feet in width, on U. S. Highway 277 South from the City of Lawton.

The soils survey of the completed grade, made prior to the preparation of plans for surfacing improvement, indicated quite uniform extents of Type A-6 and A-7 soils on rolling grade line. There appeared to be no possibility of high-ground-water, and no variation in soil strata could be found which would likely produce side-hill seepage, and necessitate extra thickness of base or special drainage.

The Department believed it desirable to place in this type subgrade a higher type base course than soil-bound gravel or stone, or one which would likely provide ample stability for the medium to heavy traffic which this highway carries. Since the gravel or stone base courses would require a bituminous surface course, it was decided to construct the Macadam type aggregate penetration base in two courses, and use this construction as a finished surface during its economical life as such, then complete this with a dense graded surface course. This seemed desirable, also, from the fact that a large commercial limestone quarry was located only 12 miles North of this project, and, many other sections of the State could be economically supplied with crushed stone should this type of "Stage Construction" prove successful.

The only work of Asphalt Macadam penetration type construction had been the 3 to 4 foot widening of existing 18 foot pavements at a few locations adjacent to the larger centers of population. This work was done in all cases by State Forces, and would not, of course, afford us the necessary experience as compared to full width roadway construction.

The specifications for this project, therefore, were not a result of our knowledge of the detailed requirements, or developed by our experience, but were selected from the standards most commonly appearing

in State Highway Department specifications, and as recommended by the Asphalt Institute, and the Bureau of Public Roads.

Macadam-type aggregate was to pass 2½" square, 90%+ passing the 2", 5 to 25 passing the 1½", and not over 5 passing the ¾". The key rock was passing the 1" square and graded uniformly to a maximum of 15% pass the No. 4, with the cover chips starting at ½" and not more than 5% passing No. 10. The binder material was asphaltic cement 85-100 penetration. The work to be constructed in two courses with the usual construction methods of leveling, rolling, applying A. C., filling or choking with key rock, and rolling and shooting the first course. The same procedure for the second course, with the addition of cover chips and finishing the surface with a minimum of ¼" variation on a 10 foot straight edge. Side forms were to be provided until work was satisfactorily advanced that they might be removed and the earth shoulder rolled into the edge of the completed base.

A number of contractors submitted bids, which were based on quotation from three different sources of aggregate available near the work, and several sources for asphaltic material. The contract was awarded to the R. R. Ryan Construction Company of Oklahoma City.

When shipments of the base rock were to be made difficulties developed at the quarry. Our plant materials inspector and the producer found that only a limited volume of this type aggregate could be produced. Some deliveries were made to the job, with slight excess passing the 1½" and the ¾" while further efforts were being made to adjust the equipment to increase production. During this time a complete new set of square mesh screens were ordered delivered by express and installed in an effort to produce adequate quantity of materials. On further runs it was evident the character of the stone and crushing equipment could not produce more than 250 tons of aggregate per day with 25% or less passing the 1½" square sieve.

The Contractor could not expect to keep a construction crew available for such small production. To stock pile the total quantities required and then proceed with placing the aggregate and constructing the road would have carried the work late into the fall. This, of course, was not contemplated and would mean excessive delay and expense to the Contractor, and the stone producer, and the Highway Department, and with such difficulty in production of aggregate, the State Highway Department could not hope to develop this type of construction for general use.

The question then arose as to whether or not it was absolutely necessary to the success of the work that this grading for Macadam Aggregate be used. It was decided to make a field investigation of this feature. It was our assumption that the coarse grading of the aggregate specified was to permit adequate penetration and coverage by the Asphaltic Cement. We believed that a reasonable increase in fines, possibly down to the ½", such that ample production could be developed at the quarry, would not, by comparison, be inferior in any respect to the coarser material as specified.

A decision was finally reached with the Contractor, Producer, and Engineers, to place and complete on the road a half-day's production each of two gradings of aggregate with increased fines. These sections to be completed in every respect as the coarser aggregate, including the key rock, and that upon completion an examination, and tests if necessary, would be made of the completed work, to determine:

1. Whether a sufficient quantity of Asphaltic Cement could be introduced into the finer graded material by penetration.
2. Whether or not such application at the approximate quantities specified would provide ample or comparable coating or coverage of the aggregate.
3. Whether the key rock as specified would be satisfactory.

Upon completion by the Contractor of the work with the two additional types or gradings of aggregate, Engineers of the Highway Department and the Bureau of Public Roads made a thorough examination. The Contractor's and Resident Engineer's organization indicated that the application of the asphaltic cement was not appreciably affected. Examination through the thickness of the work in the several gradings, with the same rate of application, showed that with the base aggregate of finer grading, only a slight apparent reduction of coverage of the aggregate was indicated. It was also apparent that application of key rock would not be affected materially as to quantity of methods of handling, but would be improved with slight increase in fines.

It was, therefore, decided that the project would be satisfactory with a change in specification of the aggregates to permit a satisfactory rate of production, which would in turn, permit the Contractor to proceed with the construction at a normal rate. The change in aggregate grading was requested and approved for base rock and key rock to the following:

	COARSE AGGREGATE		KEY ROCK	
	Original Planned Per Cent	Change to Per Cent	Original Planned Per Cent	Change to Per Cent
Passing 2½″	100	No Change		
Passing 2″	90-100	No Change		
Passing 1½″	5- 25	30-60		
Passing 1″	———	——	100	100
Passing ¾″	0- 5	0-15	95-100	100
Passing ½″	———	——	65- 90	60-100
Passing ⅜″	———	0- 5	35- 65	30- 70
Passing No. 4			0- 15	———
Passing No. 10			0- 0	0- 5

Thereafter no delays were had in production and delivery of aggregates and satisfactory progress was made by the Contractor. The work was completed on July 14, 1938.

Close records were kept and furnished us by Major W. P. Danford, Resident Engineer on the Project, and indicated that the planned quantity

of bituminous material of 0.6 gallon per square yard per inch, or 1.8 gallon per square yard on each course of coarse aggregate was approximately the right amount for the finer graded material, whereas, on the preliminary work with the coarse graded Macadam aggregate, a considerable increase in asphalt was used in order to approximately fill the voids. With exception then of the approximately 72 Stations of first course, and 65 Stations of top course completed with the original specification aggregate, the project was completed with the finer graded material. The record of the quantities of aggregates and asphalt for this part of the work compared very closely with the original estimate of quantities.

Approximately 10% overrun in base rock was due to some loss by compression into subgrade by heavy rollers. Various rates of application of key rock is shown by the field records at from 25 to 40 pounds per square yard, cover chips at 25 pounds as specified.

The base rock used averaged slightly more than 50% passing 1½" with the maximum 66%, and in this case 8% passed the ¾" and only 2% passed the ½". Key rock averaged in the limits of the specifications with less than 3% passing the 10 mesh. Cover chips averaged close to the upper limit of 35% passing the No. 4 and averaged very close to 2% passing the No. 10. Asphaltic Cement tests of individual cars showed the lowest penetration to be 86 with the highest 97. Test for penetration of material coming through the distributor at about 325° F., indicated no change in character. After 30 days the penetration was found to be 59.

The project records show that the subgrade was well shaped and thoroughly compacted. Much delay was encountered in the first six weeks due to rains, making it impractical to lay base rock thereon, and long sections of finished base had to dry out before the second course could be applied. Rains on completed sections indicated the specification seal would not keep the water out of the pavement, but the base seal of asphalt appeared sufficient to prevent any objectionable saturation of the earth subgrade. Since deep frost is a very rare occurrence in the area it is possible this will cause no appreciable loss in stability. Recent soundings after several weeks of dry hot weather indicated pockets or pools of free water in the lower base, on the sealed and dry subgrade, and the same condition on the upper course where surplus asphalt penetrated to and sealed on the top of the first course.

No other unusual conditions were noted during the progress and completion of the work. The construction methods as set out in the specifications were followed closely, and every possible effort made with modern equipment available, to secure full compliance with the specifications as to surface finish on both the lower and the top or finish course. It is evident, however, at this time that some important detail, method, procedure, or piece of equipment was left out, since it is found the smoothness or riding qualities of the finished project are very unsatisfactory. We believe this is not in any measure due to carelessness on the part of either the Engineer's organization or the Contractor but probably due to lack of skilled workmanship with which none of us were experienced. It could be a lack of thorough "keying" and "spot-leveling"

of the base rock under rolling; that the base rock was not rolled or keyed to stability; that key rock was introduced in too great an amount and an unequal application, or rolling of key rock undertaken before asphalt had cured or set up sufficiently, developing a lubricating or shoving effect under rolling which could not be worked out with key rock or cover chips. Any of these, or a combination of them might be responsible. We are inclined to the conclusion that this type of construction and our general construction methods should produce a satisfactory finish capable of serving the traveling public over a period of several years with very little maintenance cost. It is apparent that we must develop the construction technique for certain of these details which call for experienced operation of machinery or equipment, or handling of materials, that is often so necessary to success in many types of construction activities.

Whether more of this type will be constructed in Oklahoma will, of course, depend largely upon future behavior of this project and its maintenance cost.

SECTION IV

Maintenance of Bituminous Surfaces

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Livingston County
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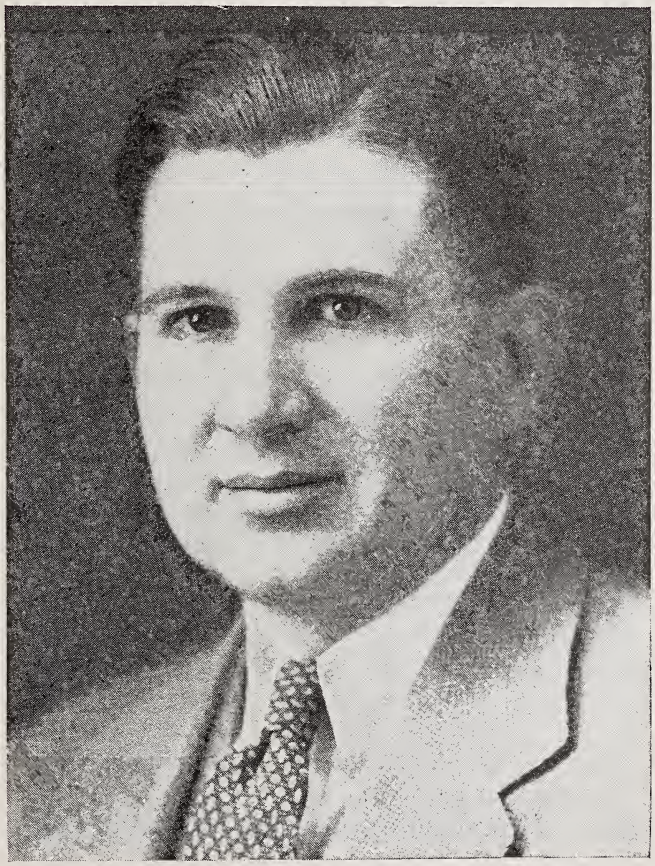
A Review and Condensation of the Papers Submitted by the Committee on Maintenance

By

A. B. NUSS,
State Highway Engineer
Kansas State Highway Commission
Topeka, Kansas



(1) MAINTENANCE ORGANIZATION — GANGS VS. SECTION CREWS.

Mr. Forrer, Maintenance Engineer for Virginia, has expressed the opinion that there is a place for both gang work and section crew or patrol units.

Gang maintenance is, according to Mr. Forrer, best suited to roads not highly improved, that require such work as replacement of materials, ditch work and gradual improvement of the surface. One objection to the patrol system is the inefficiency during periods when the weather is dry and the roads smooth. By combining five or six patrolmen into a gang during such periods of inactivity, pits and crushers may be operated

and much work performed which would be impossible with an exclusive patrol system. These gangs can be returned to small units when the weather and road conditions demand.

The section crew is best suited to higher type roads which have been properly constructed, each crew taking care of about a 60 mile section.

Mr. Forrer believes that considerable latitude should be given the Maintenance Engineer to set up the orginization best suited to the conditions.

The Montana organization, according to Mr. W. J. Leary, Division Engineer at Butte, is organized with both section crews and gangs. The sectionmen handle all regular routine maintenance over their definite section.

During the spring and summer months, special crews are sent throughout the state to do different types of special work.

In this manner, the advantages of both organizations are secured.

Mr. Reindollar, Assistant Chief Engineer of Maryland recommends a combination to suit conditions with section crews for routine maintenance and gangs for extraordinary or special maintenance such as resurfacing of sections too small to contract.

Conclusion: Both types of organization apparently have a place in a maintenance department with work of the special gang most often supplementing that of the sectionmen or crew.

(2) PROPER DECISION AS TO WHETHER TO PATCH, SEAL, REMIX, RETREAT OR REPLACE BITUMINOUS SURFACES.

Proper decision as to whether to patch, seal, remix, retreat or replace bituminous surfaces answered by five. From the answers it appears to be well agreed that an analysis of figures and the extent of necessary repairs are the primary factors in determining the type of repair.

Mr. W. J. Slavin, County Highway Engineer for Livingston County, Michigan believes that finances and economics determine largely the repairs to be made. Patching is permissable within reasonable limits, but where excessive, either the thickness of the surface should be increased or the stability of the base increased to carry the load.

Mr. F. H. Klietsch, Maintenance Engineer for Nebraska divides the factors which will affect the decision as to time and method of repair into two classes: direct, or those based on an analysis of actual conditions, and indirect, or those based on finances and facilities available for making the repair.

As an analysis of actual conditions, Mr. Klietsch recommends that an investigation of the cause of failure be made before repairs are made, as to whether subgrade, base or surface is the cause of failure. Determination of the true cause of failure, and its correction, will eliminate, to a great extent, repeated failures after repairs have been made.

Mr. D. D. Williamson of the Asphalt Institute states that four main factors should be considered:

1. Earning capacity of the road.
2. Available finances.
3. Condition of base.
4. Surface and percentage needing repairs.

On the latter, Mr. Williamson believes that, in general, a general repair is necessary when 25% or more of the surface requires attention.

Mr. S. M. Rudder of Missouri outlines the factors affecting a decision as follows:

1. Classification as to base or surface failure.
2. Alignment factor, permanent or temporary location.
3. Influence of the season in which repairs are to be made.

Mr. Rudder believes that patching is one of the most expensive types of work, due to the hand labor involved. The extensiveness of such repairs is therefore limited.

Conclusion: It appears to be well agreed that an analysis of the cause of failure should be a primary factor in determining the type of repair.

After the analysis, the type of repair will depend on cost and finances available, including the earning capacity of the road, whether a permanent or a temporary repair due to possible relocation or realignment is desired, the facilities in equipment available for repair and also the influence of the season in which the repair is to be made.

(3) METHODS OF PATCHING.

(a) Pre-mix vs. Mixed in Place on heavy mats and light surface treatments.

Seven answers received.

Mr. Stanley, Construction and Maintenance Engineer for Idaho favors pre-mix because of less danger to public and workmen while placing, and because one skilled workman can produce enough material to keep several others busy.

Mr. Dwyre, Assistant State Highway Engineer for New Mexico favors pre-mix due to difficulty in obtaining proper mix in mixed-in-place. A stockpile is usually made at construction and used for repairs.

In Nebraska according to Mr. Klietsch, all patch material is pre-mixed by blades or traveling plant, with a stockpile of mixed materials made at time of construction.

Alberta Province, states Mr. A. L. Tregillus, Bituminous Engineer, uses pre-mixed for better control.

Mr. Leary says Montana used pre-mix material.

Mr. D. D. Williamson, District Engineer for Asphalt Institute lists advantages of pre-mix as safety to public during repairs and easier to patch large areas.

Maryland apparently has been successful with both plant mix and mixed in place patches.

Conclusion: Pre-mix is considered generally to have a number of advantages over mixed-in-place. Among these are:

1. Safety to public and workmen.
2. Use of stockpiled material from construction.
3. More satisfactory material, better control of mix.
4. More economical to handle for large patch areas.

Several replies noted that the patch material must be the same as the original construction as to aggregate, and to a certain extent, the asphalt used.

(b) Type of Bituminous Binder for Patching.

Idaho patches in kind—SC3 for road mix, MC4 for plant, SC3 where material is stockpiled.

New Mexico uses SC3 in colder sections of the state and in warmer sections uses MC.

Nebraska favors MC and SC for patches. It is noted that SC with absorptive aggregates becomes dry and unusable after a short time, although with non-absorptive aggregates it is satisfactory.

Montana is now favoring SC with RC1 as a prime.

Mr. Williamson of the Asphalt Institute recommends RC for pre-mix and regular paving cement for hot patches.

Maryland uses RC2 for cold patches aging material 24 hours in stock pile before using.

Conclusion: Seven replies to this question were received. Three of these were using both MC and SC, either to patch with an oil of the same type as in original construction; or using an SC in colder regions, with an MC in warmer regions; or investigating the relative merits of the two types. One reply definitely favored MC and RC for patching, although noting the advantages of SC for stockpiling. The fifth reply favored RC and penetration asphalt. The sixth was definitely in favor of SC, while the last definitely specifies RC2.

It therefore appears that SC, MC and RC have definite advantages, depending on conditions of stockpiling, weather and probably the skill of the workmen. SC apparently possesses some advantages over other materials, which tend to be offset by the difficulty of mixing in the proper proportions with the labor and equipment available.

(c) Methods of Mixing Patch Material.

In Idaho the material is usually mixed on a suitable piece of ground

or abandoned roadway by motor graders. Occasionally a concrete mixer is used. They are trying to secure a number of small portable plants of about 100 ton daily capacity.

New Mexico uses motor graders, occasionally concrete mixers.

Nebraska uses blades or traveling plant.

Montana uses blades and patrols. Are also using small pugmill with dryer and mixer, with which each division is equipped. This is more expensive than blade mixing.

Mr. Reindollar of Maryland recommends stationary plants for large quantities, portable mixers for patching.

Conclusion: Mixing by blades and patrols is still the most common method in use. Occasionally concrete mixers are used.

There appears to be a growing tendency toward small portable mixing plants by which patching material may be mixed, especially under adverse weather conditions. Montana and Maryland are using plants of this type and Idaho is contemplating their use. There is a need for equipment of this type.

(d) Length of Time Material May be Stockpiled.

Seven answers.

Idaho expresses a limit of two years on stockpile. They seal the outside of the stockpile with SC or a cutback, or an alternate of a three inch sand blanket.

New Mexico uses in colder sections SC3 piled for over 18 months. In warmer sections, MC is stockpiled 3 to 6 months, but gets hard.

Nebraska says MC is stockpiled and is good up to four years. SC with a non-absorptive aggregate may be good up to seven years.

Alberta says MC is stockpiled up to ten days while SC has been used up to two years, although the material had hardened to a certain extent.

Montana says some SC materials mixed and stockpiled in 1930 are still usable.

Mr. Williamson of the Asphalt Institute advises that Oklahoma has stockpiled SC for more than a year, cutbacks and emulsions for 30 days.

Conclusion: The stockpile time of SC material apparently is more or less unlimited, periods up to seven years being reported with the material still usable. The quality of the asphaltic material and the protection of the stockpile probably affect this to some extent. The period of MC cutbacks is limited apparently to not more than one year, although one state reports MC usable after four years.

(e) Grading and Types of Aggregates for Patch.

Idaho uses same graduation in patch material as in mat with 100% passing the 1 inch square sieve and not over 12% passing the 200.

New Mexico conforms to original graduation except in some cases where maximum size is reduced from one inch to ¾ inch.

Nebraska uses same gradation as original material.

Motana states early gradation was too coarse with not more than 65% passing ¼ inch sieve. This was altered to 100% passing ¾, 40 to 65% passing the ¼ inch sieve and 5 to 15% passing the 200 which has given satisfactory results.

Alberta uses same gradation as original mat. Trend is to get away from densely graded aggregate, using material of which 100% is retained on a ⅛ inch circular opening.

Mr. Williamson of the Asphalt Institute suggests relatively open mix.

Conclusion: In general, the same gradation of aggregate is used for patching as that of the original construction. In some cases, it has been found advisable to reduce the size of the coarse aggregate. (The Province of Alberta reports an aggregate of which 100% is retained on a ⅛ inch circular opening screen.)

General Conclusions on Methods of Patching:

Pre-mixed material is almost universally favored for economy and safety.

Bituminous binders are largely divided into MC and SC, with some using RC cutback. Both MC and SC apparently have advantages, and in a number of states, both are being used.

Blades and patrols continue to mix most of the patching material, but there is a tendency to use small portable plants for this purpose. The latter are valuable in producing better mixes through control of bituminous binder and moisture where a dryer is used, and in general, under adverse weather conditions. One quoted advantage of the portable plant is the production of pre-mix material during the early spring, allowing more time for actual patching in more favorable weather. There appears to be a demand, both existing and latent, for equipment of this type.

SC binder has an advantage over cutbacks where pre-mixed material is to be stockpiled over a period of time. Periods of usability extending up to seven years for SC, and four years for MC have been noted. These probably represent an extreme—two years may be considered good.

An interesting note in the paper by Mr. A. D. Stanley, Maintenance and Construction Engineer of Idaho, is the protection of stockpiled material by a seal over the pile with an SC or MC, or providing an alternate of a three inch sand blanket. This brings up a possibility that by proper protection, such material might be stored indefinitely.

The grading of aggregates generally conforms to the original gradation of construction, except that in some cases, some coarse material is eliminated in the patching aggregate.

(4) SEALING.

(a) Type and Grade of Bituminous Materials Used.

Six answers.

Mr. A. D. Stanley of Idaho reports SC4 has given best results in Idaho under extreme heat and cold. Stone chips are used with this material, having not more than 8% passing the 200 screen.

For sand cover, RC4 is being used. MC was found too slow in curing to prevent pickup under traffic and also tended to become hard and brittle under low temperatures. The use of 95 plus asphaltic oil with special MC5 and RC5 cutbacks is contemplated.

Livingston County, Michigan, according to Mr. Slavin prefers a light seal of 0.25G.S.Y. of quick-breaking emulsion. 0.1G.S.Y. of MC2 with a sand cover is also used.

While in New Mexico materials of SC, MC, RC and 250 penetration asphalt are used. SC is used on surfaces which have dried out and on older surfaces. RC3 is objected to due to hardness and brittleness. MC3 is used on new surfaces and porous surfaces.

Mr. Klietsch of Nebraska says RC1, RC2, and RC3 from 0.15 to 0.30 G.S.Y. are used there, dependent on type of surface. MC2 is used on SC mats for sealing.

Mr. S. M. Rudder, Assistant Chief Engineer of Missouri divides sealing roughly into two classes, those with little or no cover, and those which utilize substantial amounts of cover. For the former both MC and RC of low viscosity are used. For the heavier type, heavy cutbacks and high penetration asphalts are used.

Conclusion: It appears that nearly all types of asphaltic materials are being used, from SC, MC, RC and penetration asphalts to quick-breaking emulsions.

One state, Idaho, has had excellent results from SC4 as a sealing material, while there is a decided increase in the use of heavier cutbacks of MC4 and RC4 grades, and high penetration asphalts applied hot. A tendency toward heavier cutbacks than the four grade is also noted in the paper by Mr. Stanley of Idaho, in which the use of a special MC5 and RC5 for sealing is contemplated.

There is a tendency to choose the asphaltic material to suit the peculiarities of each surface; if the surface is porous or loose, a material which will penetrate this surface, such as an SC, MC2 or 3 or an RC1 or 2, is used. On dense surfaces, heavier grades are used.

(b) Methods of Applying Cover Material.

Idaho recommends spreader boxes. Have also used with success a "turkey tail," a steel plate with radiating blades or fins.

Mechanical equipment and large units tend to eliminate the personal equation and results are better.

Mr. Slavin of Michigan uses homemade revolving disc spreader.

New Mexico uses sand or chip spreader, suspended or fastened to back of truck. Avoids marking surface, more convenient and safe.

Nebraska uses sand spreaders. More economical than hand spreading, more uniform.

Missouri uses mechanical spreaders to secure uniform distribution. Brooming may be desirable.

In Maryland fan spreaders used most extensively.

Conclusion: Spreaders are recommended by all of those answering this question. In some cases, hand spreading is performed, but mechanical spreaders of various types are more economical, safer and produce a much more uniform surface, and seem to be preferred. No definite mention is made of preferences for specific types of mechanical spreaders of which there are several types. There should be discussion of the relative merits of truck propelled or self-propelled rotating type spreaders as compared with box type and of agitating vs. non-agitating types. Rollers are recommended by Idaho and Missouri; Missouri recommends tandem or three wheel over five tons.

(c) Methods and Types of Plant Mix Seal Mixtures.

New Mexico has been using natural rock asphalt with 3% natural asphalt content and 2% additional bitumen. The grading is very fine and produces a smooth rubbery surface. Use 300 to 450 tons per mile with an average thickness of ½ to ¾ inch.

A tack coat of 0.10 G.S.Y. of RC1 is necessary when used as a seal.

Missouri has used an amesite mixture on two miles of old oiled mat. The seal varies from two inches over several ⅛ mile sections, to ¾ inch thickness over others. The appearance is good.

Maryland uses Maryland "C" specification similar to amesite mix very satisfactorily.

Conclusion: This type of treatment in most localities apparently is largely experimental.

General Conclusions on Sealing:

The types of asphaltic materials used for sealing are much diversified, depending on weather conditions and flexibility of seal desired, porousness of old surface and degree of penetration desired, and the type of service due to traffic and driving conditions expected. The type of cover also influences the type of asphaltic material.

There is a steady movement toward heavier asphalts for sealing, such as MC4, RC4 and penetration asphalt. These materials make possible heavier and thicker seals (amounting to armor coats or surface treatments). Riding qualities and night visibility are both improved by some types of heavy seals.

The use of mechanical cover material spreading devices is much

favored and is increasing. Much home made equipment of this type is in use, but factory produced spreaders offer many advantages in economy and standardization.

The use of rollers and dragging or brooming on cover material is recommended and used by several states.

Plant mix seals are being used by Idaho, Missouri and Maryland although the "classification" of this work as a "seal" appears to be improper.

(5) SHOULDER MAINTENANCE OF HIGH TYPE BITUMINOUS SURFACES.

(a) Rolling: Weight and Type of Roller:

In Virginia four types of shoulders are considered with type of treatment for each. These are classified as:

1. Earth from regular excavations.
2. Earth from selected materials.
3. Soil that will support grass.
4. Bituminous sealed type.

For earth shoulders of type 1, compaction with a roller is beneficial. The roller should weigh not less than eight tons. Type 2 may be compacted in like manner. Type 3, the grass shoulder, requires no other than original rolling. Type 4 is ideal (bituminous sealed type).

Mr. Reindollar would like subject discussed.

(b) Sealing of Shoulders Justified:

Mr. Forrer of Virginia feels that shoulder treatment as described is well worth while. For effective sealing of the shoulder, the material in the shoulder should be suitable. Under these conditions, sealing of the shoulder is an ideal solution.

Traffic tends to use the shoulders after treatment and wear occurs. The solution tends to be temporary.

(c) Would Saving Justify Full Width Construction:

Mr. Rudder of Missouri believes that bituminous treatment of shoulders is economically justified as well as full width construction.

General Conclusions on Shoulder Maintenance, High Type:

This question was discussed in length by Mr. Forrer of Virginia. Mr. Forrer has classified four types of shoulders and the general treatment of each.

When the soil of the shoulder is of the proper type, rolling is of considerable importance, with an eight ton roller recommended.

Sealing of earth shoulders is of temporary value when soil conditions are favorable.

If the shoulder has a good surface traffic tends to use it with resulting wear. It is inferred that full width construction may be effective and the cost justified.

(6) SHOULDER MAINTENANCE FOR LOW TYPE BITUMINOUS SURFACES.

(a) Rolling: Weight and Type of Roller:

Mr. Slavin, County Engineer of Livingston County, Michigan, believes that adequate width of surfacing and thickened edge construction of mat with a "V" channel in the subgrade at the edge, with a well trimmed berm (Shoulder slope) will minimize edge failures. Rolling is not discussed.

Mr. Rudder states that the following system is used in Missouri with excellent results:

Shoulders are built up to mat levels at the time of construction. Experiments with low shoulders (not built up) did not prove satisfactory from either appearance or elimination of edge failures. This applied to both high and low types.

Ruts are eliminated by dragging, with particular attention during winter and spring months. In the spring, shoulders are rolled with a 36 inch wide roller having a pressure of from 120 to 130 pounds per inch of width. A shoulder slope of at least ½ inch per foot is maintained. The intensive work during this time of year considerably lowers maintenance costs during the remainder of the year.

Conclusion on Rolling:

The experience of Missouri and, it is believed, many other states, indicates that rolling of the shoulders is of considerable value in reducing maintenance both of the shoulder and of the mat or other bituminous surface.

(b) Sealing Economically Justified:

In the opinion of Mr. Rudder of Missouri, the sealing of shoulders on low type bituminous surfaces is economically justified, in that with each increase in width of primed and surfaced base, maintenance work along the edge has decreased. Sealing the entire width has produced a noticeable effect in reducing shoulder maintenance.

The shoulder may be protected by either oiling or an armor coat.

Conclusion: It will probably be agreed by many that protection of the shoulder surfaces will lead to decreased maintenance through less shoulder erosion and edge failure of the bituminous surface.

Present methods of penetration alone do not always work satisfactorily, but through soil selection or stabilization with aggregate and/or oil, or by placing an armor coat on the shoulder as suggested by Mr. Rudder, very serviceable shoulders may be produced which will yield good returns in decreased maintenance of the surface.

(c) Would Saving in Maintenance Justify Constructing Surface Full Width of Roadbed.

Mr. Rudder from Missouri believes that full width construction would create substantial savings in maintenance and justify the additional cost and time.

(7) RECONSTRUCTION OF FAILED SURFACES

(a) Where Failure is Due to the Surface Itself:

Mr. Klietsch and Nebraska classifies two groups for reconstruction: one with little or no special maintenance; the second with considerable special maintenance. The first group appears to be primarily those of surface failures. The treatment given is to tear up the surface and blade it to one side, allow the subgrade to dry, correct oil deficiency in the surface material, adding RC2 to SC mat and MC to MC mat, and then relaying.

Mr. Root, Maintenance Engineer for Iowa believes that most failures occur in the base, and that the base difficulty must be completely cured before a permanent repair can be made.

No comments on reconstruction proper.

According to Mr. Rudder of Missouri, reconstruction of the surface should be made in accordance with the cause of failure. More bitumen may be added to dry mats and bitumen in mat may be reduced by addition of the proper dry aggregates. Difficulties in gradation should be corrected with additions of proper materials.

Conclusion: Failures of the surface course may be corrected, in general, by supplying the deficient factor responsible for the failure. Where the base or subgrade is not at fault, it is indicated that a reprocessing of the mat is favored as the correct procedure. In some cases increased thickness may be necessary (See Maryland). There are limits, however, to the extent to which such reconstruction can be carried.

(b) Where Failure is Due to Subgrade Distortion or Shifting of the Base:

In Nebraska, surfaces which have failed due to the subgrade, are removed from the roadway and either stockpiled or windrowed. The base is then stabilized.

The stabilization method used is determined by an analysis of the subgrade composition. In most cases, a stabilized gravel mat is constructed using new materials. From three to five inches of stabilized gravel base is constructed on A6 or A7 soils. No definite formula is used to determine the thickness; engineering judgment is depended on.

Bituminous stabilization is used on sandy and silty soils economically suited for this type. The depth of this form of stabilization depends on the quantity of the material in the subgrade soil that passes a 200 mesh sieve. Three inches are sufficient where 20% to 40% passes the 200, and four to five inches are used where 10% to 20% pass the 200.

When mats are reworked, the gradation and oil content is corrected if necessary, and the mat relaid. Stronger mats are secured from old SC mats by addition, where necessary, of an RC cutback.

Where the subgrade material is sandy and contains insufficient fines, the fines are added to increase the stability. In some cases, sandy soils may fail, due to loss of moisture and can be corrected by restoring a normal moisture content (3% to 5%).

Iowa suggests correction of the difficulty causing base or subgrade failure and mentions in particular excess loading.

Mr. Rudder of Missouri lists as causes of subgrade failure, (1) surface water and (2) underground water. The former is corrected and cured by restoring the subgrade to proper condition and then protecting by seals, the surface and shoulders. Ditches are cleaned to facilitate rapid run-off.

Failures due to underground water can be corrected if the source of the water is found and drained. If this is not possible, the faulty subgrade material should be removed and replaced with proper material, or the area bridged by a heavy base. The latter is usually more practical.

Mr. W. J. Slavin of Michigan has reconstructed failed surfaces by removing objectionable material from frost heave areas, and then scarifying the old surface and base to a depth of six inches. One and one half gallons, in one case of SC3 and in another MC2, were added to the scarified material and mixed with blades. The mixed material was spread and rolled by trucks in two lifts with a final rolling with a seven ton tandem. The reconstruction has not been down for a sufficient time to draw definite conclusions.

Conclusion: Reconstruction methods vary with the locality and cause of failure.

Nebraska and Missouri, in their comment on reconstruction, indicate that the surface material is removed (and salvaged), the subgrade placed in proper condition and the surface material replaced, applying whatever correction of oil or aggregates is necessary to the surface material.

Mr. Slavin of Michigan describes a method of incorporating the surface material with subgrade material, plus a quantity of oil, forming a stabilized or semi-stabilized subgrade which is then given a light surface treatment.

Nebraska utilizes several methods of correcting subgrade difficulties, varying from simple correction of moisture abnormalities to more involved gradation correction for bituminous stabilization for sands and stabilized gravel mats.

Missouri also salvages the bituminous surface material which is relaid after correction of the subgrade difficulties.

Correction of subgrade difficulties are made by removing the cause of the difficulty, usually excess water, or by either removing objectionable material or bridging such objectionable material with a heavy base.

Kansas has engaged in a heavy reconstruction season, in which the existing surface has been mixed with the subgrade and the whole sheepsfooted to a stable compaction, after which a penetration-armor coat has been applied.

GENERAL CONCLUSIONS

The methods of reconstruction vary with different localities, organizations and types of difficulties. Reconstruction of surfaces alone depends largely on correction of asphalt content or of gradation with the mixing and relaying of the surfacing material.

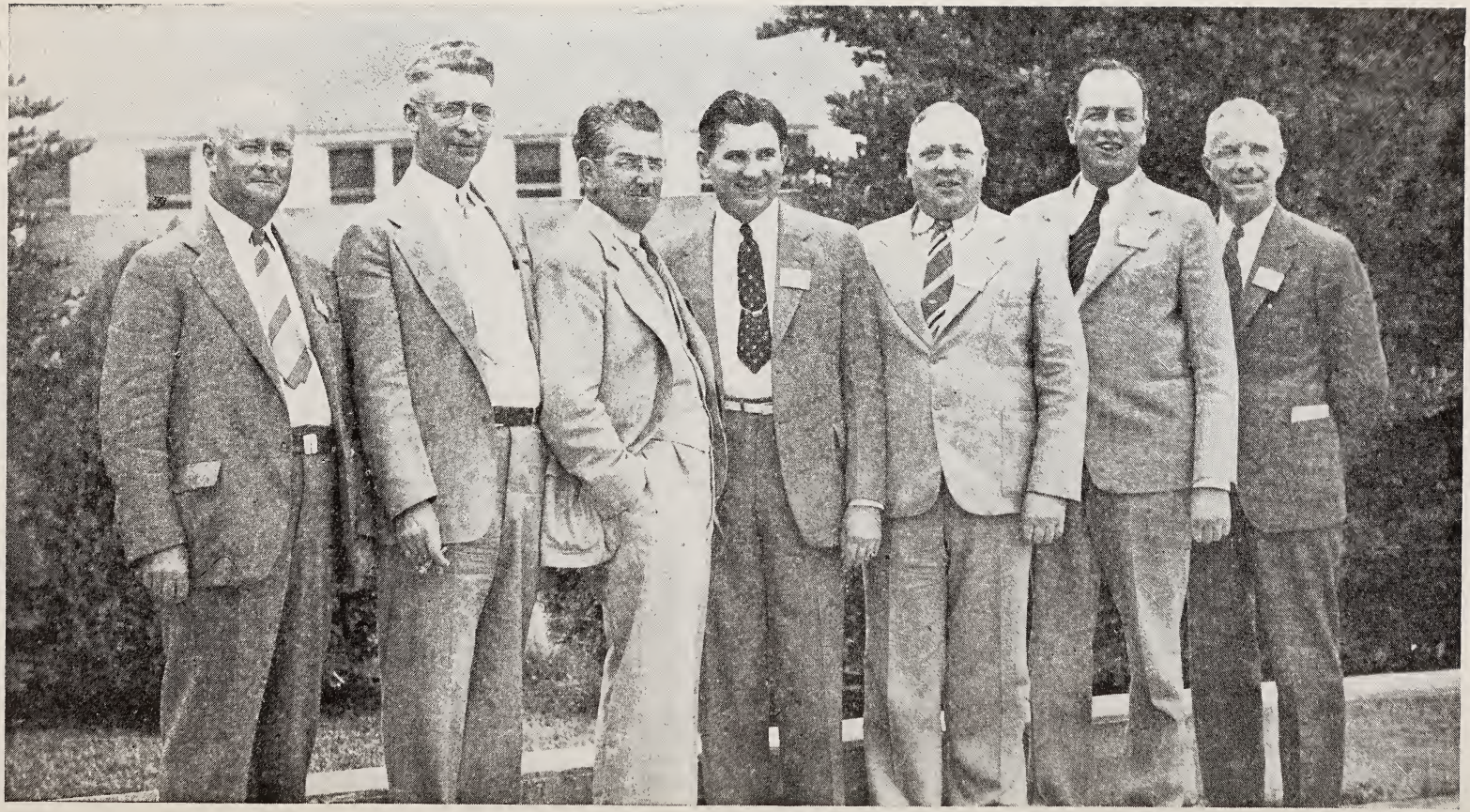
With subgrade difficulties, correction becomes more varied. The source of the difficulties, such as surface or ground water, may be eliminated as a simple solution. Sandy soils with insufficient fine material may be corrected by addition of the necessary fine material.

Subgrades with heavy soils may be stabilized to a degree incorporating the old bituminous surface, followed by sheepsfooting and compacting and then covering with a light surface treatment.

An alternate method is the bridging of unstable subgrade by heavy base construction of stabilized gravel mat, practised by Missouri.

Stabilization of soils is being accomplished by cement and bituminous admixtures. Nebraska, particularly, has used bituminous stabilization for sandy soils to a considerable extent.

Some effort is being made to extend bituminous stabilization into heavy soils by the use of SC and MC, plus the stabilizing effect of aggregates from the old surfacing when rolled and compacted. Mr. Slavin of Michigan has experimented with this form of stabilization, as has Kansas. Missouri has had extensive experience with various methods of stabilization.



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State Highway Commission
Little Rock, Arkansas

Mr. Nuss: Eleven papers were submitted for review, and I have tried to condense those into one paper here that would fully cover the subject without having any personal opinions of my own in it. I have mimeographed sheets here covering the subjects or questions that the papers were written on, and I am going to pass these out so that you will know the questions that the papers cover, and as there is room on them to put any notes you care to, we can come back and have an open discussion on your notes and the list of questoins will serve as a memorandum to you.

This covers about all of the questions that were answered in the papers and, as I said, there were eleven states that wrote on these questions. I think if we have a little recess here we can come back and have some general discussion from the floor, and I would like to have any of you submit, during the recess, any questions that you have, or any topics you would like to have brought up for discussion on any question affecting maintenance of bituminous surfaces. If you will just give me those questions, I will see that there is discussion from the floor, or you can get up and introduce the questions yourselves. I think when we come back, if we can get into a good lively discussion on a number of these subjects we will all get a good deal of benefit from it, so we will take about a ten-minute recess now. (Recess)

I think we had better get on with the rest of the program, so we will get into the discussion right away.

First I want to thank the members of this committee for the papers they submitted to me. I hope I haven't overlooked anything of consequence in summarizing their papers, because they did submit some very good papers.

These men up here on the conference board will be glad to take part in any discussion on any question that you may have and I think that they can do a good job of answering most of your questions, so shoot them right to them.

We have a question submitted here. It says, "Have any states used emulsion for seal coat without cover material? Is this material superior to MC2 or RC2?"

I would like to hear from some states that have used emulsions for seal coats, especially if you have used any of it without cover material. Will someone please give us a little discussion on emulsions used for seal coats?

T. R. Perry: Iowa

Answering the question of using emulsion for seal coat without cover, I can say that in Iowa we have used a little of this material. Last fall we used one car and put it on at the rate of about .15 gallon to the square yard.

It become tacky and sticky and we had to cover it before we could let traffic use the road.

Mr. Nuss: Have you had some experience, Mr. Slavin?

W. J. Slavin: Michigan Not without cover.

Mr. Nuss: What experience have you had using it with cover?

W. J. Slavin: Michigan We have used emulsion most exclusively as a seal material. We use only a small quantity of it, from .2 to .25 gallon to the square yard, with a fine cover, about .1 to .25 with bird's-eye sand or gravel, as we call it, and that material passes the ¼ inch about 100%, and perhaps 10% to 15% of it passes the 10-mesh sieve. Using a small amount of bitumen, we can get by with about 12 to 15 pounds to the square yard, and by having the finer particles in this cover material we get a good coverage and apparently the fine sand also has one other effect. It has a tendency to prevent bleeding through. In other words, our surface will remain gray instead of turning black.

We have used emulsion, as I have said, almost exclusively, except in a few instances where we have used an MC. That is perhaps getting off the subject a little, but we have used an MC2 material. We have used more or less of that in our construction work and tried out some of it as a seal, using only a small quantity, from ⅛ to 1/10 gallon to the square yard, using an ordinary good concrete sand for cover. It is a very economical type of seal and it is one that apparently does very well.

W. W. Zass: Arkansas Arkansas has employed emulsion as seal coat material with the use of cover chips. We have used it successfully, as well as other agents, but generally the emulsion will have a higher unit cost per gallon and a lesser AC content, so it is questionable whether it is an economical procedure in relation to other bituminous agents.

Mr. Nuss: Does anyone else have anything to offer on emulsions for seals? We would like to hear from any of you that have had any experience whatever in this. If not, we have some other questions here that have been asked. Several of these should draw out quite a little discussion.

One of them is "Contracting maintenance vs. State maintenance." I don't know how many states here have contracted maintenance. I might start this off by saying that in Kansas almost all of our sealing is contracted, and all of our reconstruction that I was telling you about a while ago is contracted. Each year we have gone a little more to contracting all of our maintenance, and as we have progressed in this we have found that we are saving ourselves a little money and we feel we are

getting better jobs. It lets us organize our own state inspection much better, and we find this one thing about having a contractor do the maintenance work, that our engineers are perhaps a little more inclined to make them do it just right than they would our own group, because our own crews, of course, are interested in showing us that they can do the work cheaply, and oftentimes it is a little too cheaply—cheap in first cost but high in the long run.

I would like to hear from some other states that are doing some contracting of their maintenance, and how they are getting along with it. Has anyone on the committee done any of that? What do you do in Missouri on your sealing? Do you do that, or do you contract it?

S. M. Rudder: Missouri — Most of it is done by ourselves. We do some contracting, but most of it we do.

Nr. Nuss — Are there any states here that have been contracting most of their sealing or their reconstruction or reconditioning of mat surfaces by contract?

T. R. Perry Iowa — We like to contract as much of our maintenance as we can. The criterion that we use is that if we can make a logical, fair estimate of what the cost will be, then we consider it a contractable item and proceed with contracting. This covers seal coats of various kinds, major reconditioning or reconstruction of old mats. The patching work, miscellaneous items of maintenance, weed cutting and so on, we have not contracted. I like the proposition of contracting so far as the maintenance forces are concerned in this respect: When you have a major reconstruction section to do with your own maintenance forces, it calls in your equipment from quite a territory and your regular maintenance is handicapped, or at least alibis are offered to this affect because the equipment is tied up. Then, too, we find that if a contractor has a dozen maintainers out on the highway all strung out on reconstruction the criticism from the public is not as severe as it is if it is our own outfit, so we like to contract anything that we can contract, if we can make a fair estimate of what the cost should be.

Mr. Nuss: Thank you. I might add that on this maintenance question we are contemplating in Kansas some time this fall or winter contracting several sections as a test to try it out and see what kind of job we will get and what price we will get for contracting a whole section for cleaning of ditches, keeping the weeds trimmed and the shoulders rolled and all of that except the patching material that is necessary. We will furnish that free to the contractor, but he will have to see that it is properly used and that it will be used when he should use it.

Just how that is going to work out I don't know, but we are in the process of writing up specifications covering it and we hope to let several sections, and they will be let for a one-year period for the maintenance of everything between the right-of-way lines.

Can anyone else here offer anything on contracting maintenance? I would like to hear from some of you as to what you think about it, whether you have tried it and found it did not work. Some of us who are getting into that more and more might be able to learn something from some of you who have had some experience with it. Is there anyone here who has had any experience with it, either good or bad? If not, we will go on to the next question.

We have been having a problem in Kansas that I presume you are having everywhere else. Several of the other states have mentioned they were having it, the problem of handling the traffic when you have a reconstruction job coming along. We find there that as soon as we get into the detouring of traffic we are always headed for trouble, and I would like to hear from some of the members of the committee whether they detour on maintenance, or whether they attempt to carry everything through, regardless of the kind of construction they are doing.

What do you do in Missouri, Mr. Rudder?

S. M. Rudder: Missouri — We in Missouri carry traffic through contruction. I don't know of any jobs we have had where we did not unless they were absolute reconstruction.

W. J. Leary: Montana — In Montana in most cases traffic is carried through construction work. This is especially true through the mountainous country where the new location follows the old road. On all seal work we arrange to route traffic through the work. RC1 is used mostly for our seal work and all traffic has to be carefully protected for a few days after application due to the slippery condition of the roadway.

Mr. Nuss: The problem of handling traffic seems to be getting to be a harder one to solve all the time. People seem to want good roads but they don't want any inconvenience during construction. In fact, in Kansas this year we have had quite a campaign against detours. Of course that was on new construction, but we have had a little on maintenance, so it looks like we are going to have to go through with everything, regardless of what it costs to do it.

I have wondered if any of the other states have had trouble along that line, and what conclusions you have come to on it. Is there anyone here that has had the

same problems, and have you changed from the detour system to carrying everything through?

I would like to ask any of the committee here now, do you have any maintenance problems that you would like to have discussed here? Have you any questions in your mind, Mr. Slavin?

W. J. Slavin
Michigan

I don't think I have.

J. J. Forrer:
Virginia

One of our biggest troubles during winter weather, for patching, is wet aggregate. We store some of our aggregate along the roads in patrol sheds, but is necessary to have so much on hand it is impossible for us to keep it all dry. When the weather gets cold and we have to patch, we are confronted with the very slow process of trying to dry the aggregate before we use it. It is also quite expensive. We have been trying some pre-coated aggregates with tars and also with asphalts to waterproof them.

It just occurred to me that I might be able to get a little more information than I have been able to accumulate myself if I would bring this subject up.

Incidently, our state does not have generally distributed commercial plants in which to pre-coat material. There are five or six plants within the state, but they are so located that we do not get a very short haul to all parts of the state. We have resorted to some of the pre-coating by our own methods before cold weather came, by first drying the stone and then mixing with about 1% bituminous material.

I would like to know if anyone else has had similar trouble with wet aggregates in the winter time.

Mr. Nuss:

Does anyone care to take the floor and answer Mr. Forrer? Are there any of the committee who have anything to offer on that?

W. W. Zass:
Arkansas

I don't believe there is any processing that will assure the obtaining of satisfactory work when unsatisfactory weather conditions prevail. That is, in winter months when low temperatures and wet aggregates are prevalent, I don't believe it is generally possible to secure maximum benefits. While it may be necessary to perform the work during that period, it is more costly and probably more indifferent.

We have a state-operated central plant set-up at Little Rock where mixtures are produced, either hot or cold. In addition, in one of our eastern districts we have a similar set-up but of smaller capacity. In each of the foregoing instances, the plant is equipped with satis-

factory drying facilities. Most of our districts operate small concrete mixers for producing mixtures employing various bituminous agents, MC's or RC's, with either dense-graded or open-graded aggregates.

J. J. Forrer: Virginia

I don't know that I expressed myself clearly. I was concerned with a pre-coating that was merely sufficient to waterproof the aggregate, not to make a dense mix as you would expect in the genearl mixes that you get from the various bituminous plants, but with the idea of adding additional bituminous material as it is used along the road in a patch.

The trouble with having the denser mixes, made with, say, from five to six per cent asphalt content, is that it is so hard to store those and keep them for any length of time, and it was with the idea of pre-coating material to make it waterproof, and then to add the additional bituminous materials, that I raised the question.

Mr. Nuss:

Does anyone care to go further into that for Mr. Forrer? Mr. Slavin, here, of Michigan, has a question that he would like to ask this group, to see if someone can help him with a problem.

W. J. Slavin: Michigan

I am wondering whether anyone has made any provision for heating of patching materials for winter use or cold weather use. Even with the slow-curing oil mixtures they get pretty hard to handle in the cold weather, and what we have done in the past is to load up a couple of trucks one day, we will say, and keep them in a heated garage over night, in order to have them workable the next day. One thing that I have thought of is the possibility of putting steam coils in the storage bin, or putting the coils into a stock pile, so that the material can be warmed up just as you need it for more or less emergency patching during cold weather.

I was wondering if anybody had had experience alnog that line.

Mr. Nuss:

Is there anyone here who has any arrangements or facilities for heating the patch material in the winter before placement? Evidently everyone is just wondering about it, about as you are, and worrying along with it. That is what we have been doing, and I presume everyone else has. But if there was some way to heat the patch material it not only would be easier to load and unload but I think it would go down into the patched place much better and probably would result in a better job.

You may have a good idea on those steam coils.

You would have to do that at a central point, wouldn't you?

W. J. Slavin: Michigan — Yes, you would.

Mr. Nuss: Another question here: I would like to hear from someone who has had experience with tar seals on bituminous surfaces. I would like to hear from some of the states that have used tar material for sealing, as to how you like it compared with cutbacks or your slow-curing oils; what results you have had, and how the costs compare and the length of life compares, in your opinion, with bituminous types.

I know some of you have used tars, and I would like to hear what luck you have had with them. I think we would all like to hear from someone that can give us an idea about it. Can't someone give us an idea here of what luck you have had with it? Have any of the committee here used tars?

J. J. Forrer: Virginia — Anything I say I don't venture as an opinion and I don't mean to be biased in any way. The State of Virginia, for purchasing purposes, put tar and asphalt in competition. The tars, as a general thing run a little higher in price, but occasionally they do hit, and when they do we use them and find them satisfactory. As a general thing they are just a wee bit slower in setting up than the RC materials with which they are in competition.

Mr. Nuss: Do you use about the same amount of cover material and the same type of cover material?

J. J. Forrer: Virginia — We use the same type of cover material and approximately the same amount of tar. However, the tar producers claim that due to the fact that not so much of their material is really volatile we could use a little less of their material than we do of the cutback asphalts.

Mr. Nuss: Do you feel your results are about the same as if you used the cutbacks?

J. J. Forrer: Virginia — I would say that they are practically the same. There is a possibility of the tar being a little more non-skid, apparently, but that is my own personal opinion.

Mr. Nuss Anyone else? Can anyone tell us about the results of using tar seals?

S. M. Rudder Missouri — In Missouri we have built one or two tar mat roads, and on one of them we did use the tar seal without cover material, but there is no comparison to draw there. The question in my mind is this: Mr. Forrer said prices were approximately the same. I wondered what prices are paid

for RC material and tars in Virginia, because in Missouri our tar prices are almost double the price of the RC's.

J. J. Forrer: Virginia

I didn't mean to infer that prices generally were the same. I said that for purchasing purposes we put them in competition, and in isolated places, once in a while, the tars will hit. As a general thing they are very much higher in price than the asphalts. My recollection is that our prices last year averaged about six cents a gallon for the RC materials, and for the tars I think it was about eight to ten cents. I am just depending on my memory, because I don't recall exactly.

Mr. Nuss:

Does any of the committee have anything else that they want to bring up at this time? Does anyone here have any question on maintenance of any kind?

T. R. Perry: Iowa

In the paper that was given this morning by Mr. Nuss before the recess I noticed there wasn't anything in particular on maintenance organization. I just suspect that every state has about the same kind of an organization and probably any difference in the organization is due to your own individual method of doing this work. I want to leave this word, however, and that is, you must not think you do not have to pay very much attention to your maintenance crews. They need close supervision.

I will give you a little illustration. It is rather far fetched, and I know none of you men have had anything like it.

About six or eight weeks ago I came up to a crew that was making some patches. They were using the surface treatment method, and I noticed plenty of smoke coming off the kettle, so I stopped and asked the man in charge what temperature he was using for the MC2 material.

He said, "I am using just what you have specified."

I asked him what that was. He said, "140."

Well, it looked like there was something wrong, so I asked him if he had a thermometer. He got the thermometer out and put it in the well. He counted—1, 2, 3, 4, 5, 6, 7, 8, 9, 10—pulled it out, and it read "140."

I said, "Why don't you leave it in there longer? You can't take the temperature so quickly."

He said, "Hell, if I leave it in there any longer it will blow the end out of the thermometer!"

You probably do not have a great deal of change in your labor. Nobody quits these days. But even with the little turnover we have you cannot overlook some of the simple—what might be classified as simple—in-

structions to these maintenance forces. I imagine that if they sat through this meeting and listened to the differences of opinion in regard to the MC's and the SC's they might think that even we do not know all there is to it, so you have to be very tolerant with those men and give them information. Do not overlook the little things!

Mr. Nuss: Does anyone else have anything here, before we adjourn? I think we have covered all the questions that have been handed to me here.

W. W. Zass: Arkansas — There has been no discussion so far as to the practice among the states in using either mud jacking or bituminous patching on sections of "sagged" concrete pavement, and I would like to get some expression of general policy in that regard.

Mr. Nuss: Is there anyone here that can answer Mr. Zass' question on that? We haven't, in Kansas, used any of the bituminous. We have done mud jacking on our pavements with good results, but we haven't used any of the bituminous materials. Is there anyone here who has?

J. J. Forrer: Virginia — I don't know whether you could call our work patching, exactly, but after a concrete road has become rough, we bring up the low places with a cold type of plant mixed material, hand placed as a rule, and then follow with a surface course of plant mixed material machine laid. Average depth is about 1½ inches. Cost in Virginia for this work is $6.00 per ton laid. We also elevate our curves with plant mixed material, though not quite as extensively as they do in North Carolina. I don't believe I could risk going up to a foot. Nevertheless, we do elevate our curves. Many of the old roads were built with a crown in the curve which makes very objectionable driving. We attempt to make the road perfectly smooth, cutting the old crown out on the outside with the plant-mixed material. We have never resorted, however, to mud jacking.

Mr. Nuss: Does anyone else have anything to offer on that?

R. V. Vokac: New Jersey — I don't want to make the meeting last any longer than necessary, but I would like to ask a question that was propounded in the Fundamentals Sections, to see what the answer might be from the maintenance angle. That question is, "How do you define 'seal coat'?"

Mr. Nuss: Did you all hear the question? The gentleman would like to know how we define our seal coat. Is there anyone here who would like to tell us how he defines seal

coat in his state? How about you fellows? Do you have a definition in Missouri.

S. M. Rudder: Missouri — No.

R. V. Vokac: New Jersey — The reason I ask the question is that there was considerable concern expressed in the other section when it was noted they spoke of "seal coats" even up to three-quarters of an inch thick. Here I note in your abstract of the papers that you mentioned "seal coats," particularly of the pre-mixed kind, running up as high as two inches.

Mr. Nuss: That is right. I think all of the sealing that was discussed in the papers that was not of the pre-mixed type was merely an application of oil to the surface, and then chips of some 100-150 yards to the mile applied; but on the pre-mixed seal coat that did run, I think, in Missouri, from three-quarters of an inch to two inches, and I think Maryland spoke of some material they called a seal coat that was put on around an inch thick. It seems that the pre-mixed is the one that was running up in thickness, and the other it is pretty well agreed is just a light distribution of oil with chips.

Mr. Rudder. you were the one who told us about your pre-mixed seal. Is that the way you speak of it in your state?

S. M. Rudder: Missouri — I think that it is probably a misnomer, all right; although one of the functions of the amesite material is sealing, it is really a surface of wearing course that has been applied to correct a very bad maintenance condition. Placing this material on the oil mat cuts off the surface water, thus acting as a seal.

Mr. Nuss: It was really put on to protect the underneath surface from water, which is what a seal coat is supposed to do.

S. M. Rudder: Missouri — Yes, and to add a wearing surface over the old oil mat, I can see the troubles which confront the Fundamentals Department.

T. V. Fahnestock: North Carolina — The opinion was expressed in the previous section that perhaps the term has been much too loosely used, and that we should make some attempt to distinquish between not only the type of seal that we are using but also perhaps classify the seal further as to its purpose, whether it is merely to make a non-skid surface or whether you are truly sealing the material for other reasons.

S. M. Rudder: Missouri — Wouldn't that be a good subject for the round table discussion this afternoon? Certainly somebody in this organization could suggest a solution, or at least a

definition of sealing and resurfacing, and where to draw the line.

Mr. Nuss: I might ask, did the Fundamentals Section arrive at any conclusion on the subject?

S. M. Rudder: No.
Missouri

Mr. Nuss: Is there anything else to be brought up at this time? I again want to thank the committee members and these other gentlemen for coming up and helping with this discussion.

We stand adjourned.

Papers Prepared

on

Maintenance of Bituminous Surfaces

for

The Montana National Bituminous Conference

Maintenance Methods

By

R. M. REINDOLLAR

Assistant Chief Engineer
Maryland State Roads Commission
Baltimore, Maryland

1. **Maintenance Gangs versus Sections Crews.**

Recommend combination to suit conditions.

a. Section crews to take care of shoulders, ditches, culverts, and maintenance of right of way; cutting grass, etc., has advantage of being in closer touch with each section of road and, therefore, gives better service to traveling public.

b. Gangs for extraordinary maintenance, resurfacing short sections too small to put under contract, storm damage, frost heaves, etc.

Gangs system can be better supervised, more economically supplied, and a greater amount of work accomplished per man. Number of workmen can more easily be increased, or decreased, to meet needs of situation.

2. **Proper decision regarding treatment of bituminous surfaces—patch, seal, remix, retread, or replace bituminous surface.**

Each method will give satisfactory results in its particular place and all are necessary.

Skin patching should be done in places where the mat is breaking up over very small areas, but this type of work should be held down to a minimum. The best time is in the spring or in the fall.

If road is in good shape, except a few pot holes, then, it should be patched, pre-mix operation preferred.

If road shows no evidence of subgrade failure, possibly smooth, surface dry, then it should be sealed. This work should be done in the summer months, preferably early spring to avoid traffic on roads where mat is dead.

If surface of road is slightly uneven, due to excessive amount of bituminous material, then, it should be remixed. If surface shows evidence of being too weak and needs additional thickness, then, it should be retread.

3. **Methods of Patching.**

a. This subject covered somewhat under section two. Both plant

mix material and mix in place method for heavy mats have been used successfully in that state.

b. See mimeographed sheets — Specifications for Bituminous Materials and Suggested Uses for Bituminous Materials.

c. Properly equipped stationary plant preferably, especially where large quantities of material are being used on resurfacing operation. Portable bituminous mixers are satisfactory for patching work.

d. Do not recommend stock piling of mixed materials, although working conditions in sparsely settled areas may require this where it would not be necessary in this state.

e. See tabulation sheet on grading of aggregates.

4. Sealing.

a. See mimeographed sheets. Suggested Uses for Bituminous Materials.

b. Fan spreaders used most extensively in this state.

c. Maryland Specifications "C" similar to an Amiesite mix found to be very satisfactory. (Specifications will be forwarded to interested parties on request.)

5. Shoulder Maintenance of high type Bituminous Surfaces.

This subject could better be covered by actual discussion at Convention as a great deal can be said both for and against placing bituminous material on shoulders depending upon conditions. Necessity depends on width of paved surface of roadway in relation to volume of traffic using it, also, topography of country, nature of soil, and precipitation to determine effect of scour, which would justify excessive first coat by reduction of maintenance operations.

6. Shoulder Maintenance of Low Bituminous Surface.

Same conditions prevail as in Section Five.

7. Reconstruction of Surface Failures.

a. Where failure is due to the bituminous surface itself, the surface should be scarified, reshaped, and an additional thickness of surface placed on the old surface. Volume of traffic and probable increase in traffic should be taken into consideration. If surface failure is due to heavy loads which were not anticipated at time of original construction, higher type of pavement should be constructed.

b. Where failure is due to subgrade distortion, then, proper stabilization of subgrade and drainage corrections are apparently necessary.

Tabulation covering the principal types of aggregates used by the Maryland State Roads Commission for various construction and maintenance purposes:

S. R. C. Designation	MATERIAL	USES	GRADING REQUIREMENTS: Per Cent Passing Square Opening Screens as Indicated									
			3½	3	2½	2	1½	1	¾	½	⅜	No. 4	No. 8
Aggregate No. 1	Stone, Slag	Macadam Base	100	90–100		35–70		0–15					
Aggregate No. 2	Stone, Slag	Penetration & Water-bound Macadam			100	90–100	35–70	0–15					
Aggregate No. 3	Stone, Slag (Gravel for surface treatment only)	Macadam Chips, Surface Treatment Cover, Heavy Patching						100	90–100		0–15		0–5
Aggregate No. 4	Stone, Slag	Ordinary Patching Surface Treatment Cover							100	80–100		0–15	0–5
Aggregate No. 5	Stone, Gravel	Paving Concrete Class B Concrete (un-reinforced			100	95–100		35–70		10–30		0–5	
Aggregate No. 6	Stone, Gravel	Class A & Reinforced Class B Concrete					100	90–100		25–60		0–10	
Aggregate No. 7	Stone, Slag	Thin Patching, Seal Coat, Crack Filling	Passing No. 16 sieve–not more than 5%							100	90–100		0–15
Aggregate No. 8	Crushed Stone, Crushed Gravel or Slag	Road Mix Surface Course						100	50–85	25–40		0–5	
Aggregate No. 9	Stone, Slag, Gravel	Backfill for Tile Underdrain, only						100	90–100			0	
Aggregate No. 10	Stone, Slag	Dust for Macadam Base and Water-bound Macadam	Passing No. 100 sieve—10-30%						100	90–100			

NOTE: Aggregate for Bituminous Surface Courses A, B and C are blended at the asphalt mixing plant; for details of aggregate requirements for these items, see book of specifications.

STATE OF MARYLAND
STATE ROADS COMMISSION
BALTIMORE, MARYLAND

SUGGESTED USES FOR BITUMINOUS MATERIALS—LIQUID ASPHALT

RC-1.....Surface Treatment with light cover, free from dust. May be warmed slightly.

RC-2.....Cold Patch Mixtures, open graded material. May be warmed slightly. Should be aged in stockpile 24 hours before use.

RC-3.....For road mix wearing course roller compacted. May be heated to 150° F.

RC-4.....For roller compacted plant-mix wearing course, open graded aggregate from 1½″ to ¼″. May be heated 150° F.

RC-5.....For penetration macadam in cold weather. May be heated to 180° F.

These asphalts do not harden rapidly:

MC-1.....For prime coat for initial surface treatment, cold application.

MC-2.....For traffic compacted road mix with densely graded aggregate. May be heated to 150° F.

MC-3.....For roller compacted road-mix wearing course, open graded aggregate. May be heated to 150° F.

MC-4.....For roller compacted plant-mix wearing course in hot weather, open graded aggregate. Aggregate and asphalt should both be warmed.

MC-5.....Same as for MC-4, except that aggregate may be more open.

These asphalts do not harden and do not develop high cementitiousness:

SC-1.....Dust layer.

SC-2.....Traffic compacted road mix.

SC-3.....Traffic compacted road mix, dense graded aggregate.

SUGGESTED USES FOR BITUMINOUS MATERIALS — TARS

RT-1.....Soil stabilization and occasionally prime coat on heavy clay soils. May be heated to 125° F.

RT-2.....Prime coat on medium clay soils and for use in cold weather. May be heated to 125° F.

RT-3.....Prime coat in hot weather on rather open surfaces. May be heated to 130° F. Soil stabilization.

RT-4.....Surface treatment with open graded cover. May be heated to 130° F.

RT-5.....Surface treatment where heavier carpet coat is wanted; also, road mix. May be heated to 150° F.

RT-6.....For open graded road mix in cold weather. May be heated to 150° F. Also, soil stabilization.

RT-7.....Same as RT-6, except that the material may be used as plant mix. May be heated to 225° F.

RT-8

RT-9.....Hot surface treatment. Plant mix and seal coat. May be heated to 225° F.

RT-10.....For use in hot surface treatment on open surface. May be heated to 225° F.

RT-11.....For crack filler and penetration macadam in cold weather. May be heated to 250° H. Hot patch.

RT-12.....For penetration macadam. May be heated to 250° F.

RTCB-5.....For cold patch in cold weather; also, road mix in surface treatment (quick setting). May be heated to 120° F.

RTCB-6.....Same as RTCB-5, except for warm weather.

STATE OF MARYLAND

STATE ROADS COMMISSION

SPECIFICATIONS FOR BITUMINOUS MATERIAL

Revised April, 1937

METHODS OF TEST FOR ASPHALTS

Specific Gravity	A.A.S.H.O. T-43
Sampling	A.A.S.H.O. T-79
Flash Point	A.A.S.H.O. T-79 (Liquid) T-48 (Cement)
Furol Viscosity (Saybolt Furol)	A.A.S.H.O. T-72
Distillation	A.A.S.H.O. T-78
Float Test	A.A.S.H.O. T-50
Penetration Test	A.A.S.H.O. T-49
Ductility Test	A.A.S.H.O. T-51
Solubility in Carbon Disulphide	A.A.S.H.O. T-44
Water and Sediment	A.S.T.M. D-96-30
Homogeneity	A.S.T.M. Proceedings, 1936 Oliensis Spot Test
Proportion of Bitumen Soluble in Carbon Tetrachloride	A.A.S.H.O. T-45

METHODS OF TEST FOR TARS

Sampling and Testing of Road Tars shall be in accordance with the following Standard Methods of the A. A. S. H. O.

Sampling	T-40
Specific Viscosity	T-54
Float Test	T-50
Specific Gravity	T-43
Total Bitumen	T-44
Water	T-55
Distillation	T-52
Softening Point	T-53

ASPHALT CEMENTS

Prepared from Petroleum:

The Asphalt Cement shall be homogeneous,[1] free from water and pressure still residues, shall not foam when heated to a temperature of 175° C., and shall conform to the following requirements:

	Maryland 'A'	Maryland 'B'
Penetration, at 25° C., 100 g., 5 sec.	85-100	50-60
Total Bitumen (soluble in carbon disulphide) Not less than	99.5%	99.5%
Proportion of Bitumen soluble in carbon tetrachloride,, not less than	99.0%	99.0%
Ductility at 25° C., not less than	100 cms.	100 cms.
Flash Point, °C., not less than	200	200
Loss at 163° C., 5 hours, not more than	1.0%	1.0%
Penetration of Residue at 25° C., 100 g., 5 sec., as compared to penetration before heating, not less than	70%	70%

1. As indicated by Oliensis Spot Test (Proceedings A.S.T.M., 1936)

Prepared from Native Asphalt:

The Asphalt Cement shall be homogeneous, free from water, shall not foam when heated to a temperature of 175° C., and shall conform to the following requirements:

Penetration at 25° C., 100 g., 5 sec.	85-100	50-60
Total Bitumen (soluble in carbon disulphide) Not less than (Note 2)	95.0%	Note 3-T 60% B 94%
Proportion of Bitumen soluble in carbon tetrachloride, not less than	99.0%	99.0%
Ductility at 25° C., not less than	75 cms.	75 cms.
Flash Point, °C., not less than	200	200

Loss at 163° C., 5 hours, not more than	2.0%	2.0%
Penetration of Residue at 25° C., 100 g., 5 sec., as compared to penetration before heating, not less than	60%	60%

2. For Bermudez Asphaltic Cement.

3. T for Trinidad Asphaltic Cement.
B for Bermudez Asphaltic Cement.

LIQUID ASPHALTS — RAPID CURING TYPES

This asphalt shall be free from water, homogeneous, and free from pressure still residues. (Note 1). It shall conform to the following requirements, when tested in accordance with the methods hereinbefore enumerated:

Base stock for these asphalts shall conform to requirements for Maryland 'A'.

The Flux shall be a petroleum product and shall conform to the following distillation requirements:

Total Distillate to 200° F.....Not more than 8%
Total Distillate to 300° F.....Minimum 30%
Total Distillate to 400° F.....Minimum 90%
End Point 425° F.

General Requirements:	RC-1	RC-2	RC-3	RC-4	RC-5
Flash Point (Open Tag) °F.	80+	80+	80+	80+	
Furol Viscosity at 122° F.	80–160	200–400			
Furol Viscosity at 140° F.			275–400	700–1400	
Furol Viscosity at 210° F.					125–250
Distillation, % by volume, Total Distillate to:					
374° F.	5+				
437° F.	12+	10+	3+	0.5+	5+
600° F.	25+	20+	14+	7+	10+
680° F.	40–	35–	30–	25-	20–
Tests on Residue from Distillation:					
Penetration 77° F., 100 g., 5 sec.	60–120	60–120	60–120	60–120	70–110
Ductility at 77° F.	100+	100+	100+	100+	100+
Per cent Soluble in CS_2	99.5+	99.5+	99.5+	99.5+	99.5+

Note 1: As indicated by Oliensis Spot Test (A.S.T.M. Proceedings, 1936).

NOTE:

For products in which the asphalt base is refined from California crudes, the distillation requirements will be changed to the following:

	To 374° Min.	To 437° Min.	To 600° Min.
RC-1	4%	8%	14%
RC-2		9%	16%
RC-3		2%	12%
RC-4		0.5%	5%
RC-5		1%	4%

LIQUID ASPHALTS — MEDIUM CURING TYPES

General: The base stock of this asphalt shall conform to requirements set up under these specifications for Maryland 'A':

This asphalt shall be homogeneous, free from water and free from pressure still residues. It shall conform to the following requirements when tested in accordance with the methods hereinbefore enumerated;

General Requirements	MC-1	MC-2	MC-3	MC-4	MC-5
Flash Point (Open Tag) °F.		150+	150+	150+	150+
Furol Viscosity at 77° F.	40–150				
Furol Viscosity at 140° F.		150–250	300–500	500–800	
Furol Viscosity at 180° F.					170–280
Distillation, per cent by volume, Total Distillate to:					
437° F.	10–	2–	2–	1–	1–
600° F.	25–	10–20	8–20	16–	14–
680° F.	50–	27–	25–	25–	20–
Tests on Residue from Distillation:					
Penetration 77° F., 100 g., 5 sec.	70–300	100–300	100–300	100–300	100–300
Ductility at 77° F.	100+	100+	100+	100+	100+
Per cent Soluble in CS_2	99.5+	99.5+	99.5+	99.5+	99.5+

LIQUID ASPHALTS — SLOW CURING TYPES

This asphalt shall meet the following requirements when tested in accordance with the methods hereinbefore enumerated:

General Requirements:	SC-1	SC-2	SC-3
Water and Sediment, per cent	2–	2–	2–
Flash Point (Clevelond Open Cup) °F.	150+	200+	200+
Furol Viscosity at 77° F.	20–150		
Furol Viscosity at 122° F.		200–320	
Furol Viscosity at 140° F.			150–300
Distillation per cent by volume:			
Total Distillate to 437° F.		2–	2–
Total Distillate to 600° F.		15–	10–
Total Distillate to 680° F.	50–	25–	20–
Tests on Residue from Distillation:			
Float at 122° F.	50–	25+	25+
Per cent soluble in Carbon Disulphide	99.0+	99.0+	99.0+

ROAD TAR SPECIFICATIONS

Grades:	RT-1	RT-2	RT-3	RT-4	RT-5	RT-6	RT-7
Consistency:							
Engler Sp. Visc. at 40° C.	5–8	8–13	13–22	22–35			
50° C.					17–26	26–40	
Float Test at 32° C.							50–80
Float Test at 50° C.							
Sp. Gravity at /25° C.	1.08+	1.08+	1.09+	1.09+	1.10+	1.10+	1.12+
Total Bitumen, % by weight	88+	88+	88+	88+	83+	83+	78+
Water, % by volume.	2.0–	2.0–	2.0–	2.0–	1.5–	1.5–	1.0–
Distillation, % by wt.							
To 170° C.	7.0–	7.0–	7.0–	5.0–	5.0–	5.0–	3.0–
To 200° C.							
To 235° C.							
To 270° C.	35.0–	35.0–	30.0–	30.0–	25.0–	25.0–	20.0–
To 300° C.	45.0–	45.0–	40.0–	40.0–	35.0–	35.0–	30.0–
Softening Point of Distillation Residue, °C.	60–	60–	35–60	35–60	35–60	35–60	35–60

Grades:	RT-8	RT-9	RT-10	RT-11	RT-12	RTCB-5	RTCB-6
Consistency:							
Engler Sp. Visc. at							
40° C.....							
50° C.....						17–26	26–40
Float Test at 32° C.	80–120	120–200					
Float Test at 50° C.			75–100	100–150	150–220		
Sp. Gravity at /25° C.	1.14+	1.14+	1.15+	1.16+	1.16+	1.09+	1.09+
Total Bitumen, %							
by weight	78+	78+	75+	75+	75+	80+	80+
Water, % by volume..	0	0	0	0	0	1.0–	1.0–
Distillation, % by wt.							
To 170° C.	1.0–	1.0–	1.0–	1.0–	1.0–	1.0–8.0	1.0–8.0
To 200° C.						5.0+	5.0+
To 235° C.						8.0–18.0	8.0–18.0
To 270° C.	15.0–	13.0–	10.0–	10.0–	10.0–		
To 300° C.	25.0–	25.0–	20.0–	20.0–	20.0–	35.0–	35.0–
Softening Point of							
Distillation Residue,							
°C.	35–60	35–60	40–65	40–65	40–65	40–65	40–65

MARYLAND 'K'
(Asphalt Crack Filler)

This material shall be an asphalt of eighty-five (85) to one hundred ten (110) penetration in which there has been homogeneously incorporated fifteen (15) to twenty-five (25) per cent by weight of mineral flour of such fineness that no appreciable separation of the mixture will occur while being maintained in a liquid condition. The mixture shall meet the following requirements:

	Min.	Max.
Penetration at 25° C., 100 gr., 5 sec.	65	100
Ductility at 25° C., CM	30	
Inorganic insoluble in CS_2, per cent by weight	15	26
Evap. loss, per cent weight, 50 grs., 5 hours 163° C.		2

Maintenance of Bituminous Surfaces

By

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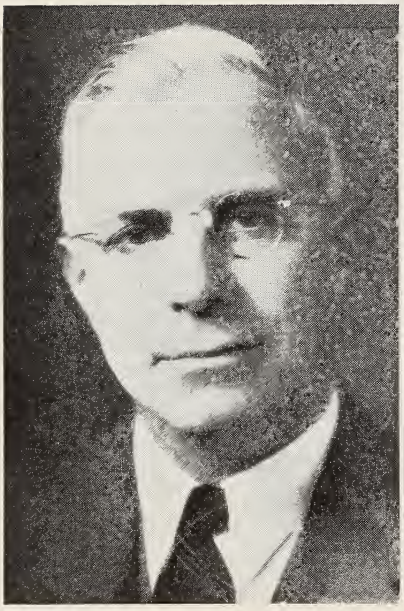
MAINTENANCE ORGANIZATION

GANG VS. SECTION CREWS



There is no controversial meaning in the above subject. There is a place in road maintenance for gang work and a place for the section crew or so called patrol unit. The kind of organization to be used is the one that will produce the best results and give the best service under the conditions found.

Gang Maintenance

This organization is best suited to roads not highly improved, that require considerable additional replacement of materials, excessive ditch work, and a gradual improvement of surface, etc.

Referring to the problems in Virginia, where there are 37,000 miles of Secondary System roads and 9,000 miles of Primary System roads to be maintained; it has been found that for gradual improvement of earth road conditions, especially where local material is available, that the gang system works out to a wonderful advantage. For instance, about all that can be done to an earth road without surface is to keep it free from ruts. This can be done by the patrol system, but the conditions recur after each rain. During dry weather and after the roads have been smoothed up, the patrolman practically marks time until the next rain because his organization is so small and equipment so light that even though he and his one or two helpers keep busy they accomplish very little.

On the other hand, if you combine say five or six patrolmen and their equipment under one head, the trucks can be more quickly loaded by the helpers with no loss of time by helpers or laborers waiting at the pit or source of material. Crushers can be operated and much valuable material produced with the gang system, where such an operation would be impossible with an exclusive patrol system.

During weather when roads need dragging or light machining, the gang is split up into small units and smooth the roads up in the area

assigned to them. This completed the gang is put back into operation and in a years' time many miles of roads are covered with materials that stabilize the earth, thereby producing year-round serviceable roads.

Section Crew or Patrol Maintenance

This organization is best suited to roads that are surfaced with materials that do not rapidly wear away under traffic and where the roads have been properly constructed with good drainage ditches. The patrolman and his helpers can usually take care of conditions on a sixty (60) mile section provided repair work is light and his attention is not confined too long at one point. If the same roads were maintained by a gang, in order to keep the force busy, the mileage would have to be greatly increased. This would cut down the frequent inspections so necessary to keep a road in first class condition. Under heavy traffic small holes soon become larger, dangerous and more costly to patch.

Conclusion

In conclusion, it is believed that the Maintenance Engineer should be given latitude to set up the organization best suited to the conditions. There is certainly a place for the various systems in every state, and it is believed that no one system will apply.

SHOULDER MAINTENANCE OF HIGH TYPE BITUMINOUS SURFACES

Shoulder maintenance for any type of surface is the one operation that has more puzzling features than any other work connected with general maintenance work. The variation of soils encountered, the amount and type of traffic using the road, the grades on which the road is built, and many other influences have a bearing on the proper operations to perform in securing the best results.

Rural roads only are considered in this article where parking is not general and drainage is directly into well defined side ditches. On such roads there are four general types of shoulders to be considered, i. e.: (1) earth type built from regular excavation material, (2) earth type built with selected materials that naturally become stabilized, (3) soil type either natural or made on which grass will grow, and (4) the bituminous sealed type.

1. Earth type shoulders built with regular excavation materials present a problem that at the best are expensive. The material dusts up in dry weather and erodes badly in wet weather. Maintenance operations to keep them well shaped in many cases tend to aggravate the above faults, and considerable expense is involved in replacement. Compaction helps some and should be done with a roller weighing not less than eight tons. The material as generally found in regular excavation will not grow grass and is unsuited for a bituminous seal coat. Stabilization with crushed stone or gravel near the edge of the pavement is necessary to prevent tracking of dirt onto the pavement and to cut down erosion. Where traffic is light the aggregate need not be applied more than several inches in

depth and 1½ to 2 feet in width. This, while costly to begin with, pays big dividends in service and satisfaction.

2. Few states have specifications that provide for selected materials to be used for shoulders that will naturally become stabilized. In sections where such material is available, this item should be given serious consideration, and included in the contract. Such material as soil with natural aggregate will become stable and if properly handled will grow grass. Natural gravel, either creek or bank, with a small amount of filler, makes ideal shoulders. Either of the above materials, as a rule, will hold bituminous seal after becoming compacted.

3. Where traffic is light and the wear along the edges is of no consequence, the grass shoulder is ideal. To produce grass it is necessary to have fertile soil. Careful selection of field soil and the use of a small amount of fertilizer will produce satisfactory results in a short time. The beautiful effect is well worth the expense. Provision for such construction should be provided for in the contract. It is quite necessary to keep the shoulders low as grass has a tendency to catch dirt and gradually build higher than the pavement. One disadvantage is that drainage is a bit sluggish through grass.

4. Bituminous sealed shoulders are ideal if properly constructed of material sufficiently strong to carry traffic. As a general thing, however, it is observed that traffic, either through carelessness or intent, use such shoulders more than necessary, thereby increasing the wear and bringing about high maintenance costs. In congested areas and on heavily travelled roads where any other material will not stand up, they perform a temporary service that is well worth while until widening of the pavement is provided.

As stated in the beginning of the article, different conditions will permit different types of shoulders, and it has been the endeavor to explain to some extent the use of each type of shoulder. It is believed, however, as indicated that more care by the construction department should be given to shoulders by the use of proper materials. This seems to be one item that is generally ignored by designers of highways, leaving the burden on the maintenance departments to correct what is in reality faulty construction.

Maintenance of Bituminous Surfaces

By

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THE SEALING OF BITUMINOUS MATS

The following remarks in regard to the sealing of bituminous mat road surfaces apply only to Idaho and the intermountain Rocky Mountain region where climatic conditions are similar and where a large volume of roadside material is readily available for surfacing and where, to a large extent, the haul on oils is long.

Seal coating in the opinion of most of those engaged in providing bituminous surfaced roads has the same relation to oil mats as a roof has to a house. It seals off the moisture, prolongs the life of the structure, prevents the action of frost within, deflects water to its proper drainage channels, prevents distortion and displacement and tends to stabilize the foundation.

The purpose coincident with seal coating is the development of non-skid road surfaces in relation to the tractor braking form of motor vehicles.

The use of rock chips in conjunction with sealing affords a non-skid surface. The size of the materials and the adhesion and quantity and quality of the oil binder makes the difference between an oil and a rock wearing surface. The former tend to glaze and the other is non-skid and affords the maximum adhesion of tires consistent with smooth riding.

There has been much said and printed about the application of seal coats. Both good and bad results have been achieved and expounded upon by those having used similar procedure. When a result is disclosed, the reasons given for the differing results from similar procedure are usually too intangible for the development of a more precise formula. In other words, the personal equation and many other equally intangible variables such as local and temporary conditions intrude themselves at various places and times and in varying quantities. The result is irregularity in the finished product. A closer understanding of these variables is one of the purposes of these meetings of the National Bituminous Conference, which are, in a sense, experience meetings between the various states.

(a) **Grade and Type of Materials.** Some of the best results have

been obtained from using SC-4 oil. This is not considered orthodox practice by many authorities. This oil, however, is more pliable, especially in the intermountain country where there is extreme heat and cold. It does not get as brittle as the medium curing materials.

The results obtained were secured by using stone chips of a size passing a 5/8 inch square opening with not more than 8% passing a 200 mesh sieve.

This oil, however, is not recommended for use with sand or other fine materials. There seems to be a tendency to over-apply sand with the result that a scale is formed which abrades under traffic and tends to corrugate the mat proper.

The sand, as a whole, does not prove as dependable as rock chips for the reason that even if a good result is obtained a glazed finish, dangerous to traffic, soon develops. Sand, if used, should be applied in two coats to permit more exact application.

In past years MC-4 has been used largely for sealing but has resulted in a very considerable loss of cover coat material due to the pick-off of fast traffic and wide tires. This has been aggravated by the slow curing qualities. The older methods are passing and newer ones developed to meet the increasing demands of traffic.

A large percentage of sealing jobs this year will be done with RC-4 oils to avoid the losses of material resulting from previous methods. The excessive range in temperatures in the intermountain country requires rapid curing of seal coats to prevent the loss of oil through too great penetration into the oil mat and lack of bonding of the cover material. This is particularly true in sealing new mats.

The amount of oil applied for satisfactory results is about .25 to .3 of a gallon per square yard using 25 to 30 pounds of cover coat material per square yard. Allowance, however, should be made for loss in handling. Penetration can be delayed and a more effective application of oil to rock chips effected by lightly spraying the mat with water immediately ahead of the application of bituminous materials. Very satisfactory results have been obtained by this method. Operations are thus enabled to be carried on throughout the day instead of being stopped by excessive temperatures which cause unduly rapid penetration.

When the spray dampened surface of the oil mat is covered by an application of oil a slight boiling effect is produced. This action tends to hold up the oil and delay penetration until the chips can be placed, thus allowing time for the chips to enter the oil before it penetrates or congeals. This method is particularly effective when used in sealing new mats.

Some of the Intermountain states use 95+ road oil almost to the exclusion of other kinds. As parts of this area are far from a base of oil supply the use of 95+ is economical from the standpoint of transportation costs.

Changes in practice have been put into effect more rapidly in some states than others. Cutbacks have been used exclusively by some states

because they are easier to handle than the heavier asphaltic oils such as 95+. However, the increasing demands of traffic are developing conditions that are forcing recognition of the fact that practice must keep in step, or inaction will cost more than action. It seems apparent that all of the intermountain states will probably adopt heavier sealing materials, that will afford a more substantial non-skid surface through the use of large rock chips more firmly embedded in heavier asphaltic oils. Idaho, for example, may in the near future adopt a standard requiring the use of 95+, MC-5 and RC-5 oils.

(b) **Methods of Applying Cover Material.** The proper distribution of cover material is a considerable factor in reducing costs through reduction of losses of material. The arrival of effective mechanical spreaders on the market is of comparatively recent date and this equipment not only effects a saving in material but in time also. For the best results these are equipped with agitators and operated with 3½ to 5 ton trucks because they permit a longer and more economical spread of material.

Satisfactory results in spreading can be obtained by various of the homemade rigs which preceded the present mechanical spreaders but it is difficult to standardize homemade equipment.

Spreader boxes have been used to fairly good advantage and also what is known as a "turkey tail." The latter consists of a steel plate with radiating vanes attached to the rear of a truck. The average homemade rig is most effective when not over ten feet in width. Although the homemade boxes and spreaders were rather crude they answered the purpose for a long time though now it is evident that they were responsible for some of the irregularities that we had not attributed to them. In the past, also, spreading for surface was not the customary practice that it is today and this probably accounts for the lag in development of mechanical devices.

Mechanical equipment and large units tend to remove a little more of the personal equation from the results secured.

The rolling of the rock chips after they have been spread upon the mat is a very essential factor in securing results in both sealing and producing a lasting non-skid surface. Rolling should immediately follow the spreading of the rock chips in order to compress the chips into the oil before it penetrates the mat or congeals and also to bed them into the mat to some extent.

There is a relation between the hardness of the chips, the mat, and the weight of the roller used. Excess weight of roller has a tendency to crack the chips thus reducing their size and reducing their effectiveness in producing the non-skid surface they were expected to create. This cracking or crushing might also break the sealing.

Different states vary in their preference for tandem, three-wheeled and pull rollers and between five and eight tons of weight. A three-wheeled power roller will cover the greater area in a given time because of its additional width. There is little unanimity of opinion on this subject, however.

During the past year Idaho used a pneumatic tired roller of adjustable

weight with very satisfactory results. This machine is fast and is an all-year piece of equipment and can be propelled by most any type of equipment. In my opinion this type of roller has many possibilities.

(c) Have no comments to make on plant mix seal mixtures.

METHODS OF PATCHING

(a **Pre-mixed vs. Mixed in Place. Heavy and Light Treatments.** Our fast and irresponsible drivers are the first factor to be thought of when considering methods of doing work on the traveled way of our highways. The driver to some extent is responsible to himself but we are responsible for our men. However, the driver and the methods we require our men to use, determine the extent to which our men and the public are exposed to danger. They determine also the amount of time that the men spend in protecting themselves. The methods used also determine the amount of equipment utilized and exposed to traffic.

These requirements all point to the use of methods of patching that require a minimum of exposure of men and equipment on the traveled way. Pre-mixed patching meets these requirements to a greater degree than mixed, in place, patching because a greater part of the preparatory work is done off the traveled way.

Another factor often disregarded when methods are under consideration is the amount of education required in the personnel in proportion to their permanence in service. In other words, simple, direct methods requiring a minimum of experience must be employed when the personnel is too liquid to justify extensive education. Re-mixed patching has a further advantage in this respect also as one experienced man doing the mixing is sufficient to keep several unskilled men busy placing the patch of pre-mixed material.

No distinction in Idaho is made between the methods used for light and heavy oiled mats except that on light mats the patch material is placed a little deeper than the original mat.

(b) **Type of Bituminous Binder for Various Patches.** The type of binder is governed by the type contained in its surface being patched. We generally use SC-3 on patches for road mix jobs and MC-4 for plant mix and pavement, thus adhering to a practice of patching in kind.

A distinction, however, is recommended in regard to material that is stockpiled and which may be left in stock for some time. In that case, slow curing bituminous binder has specific advantages. There is less evaporation of volatiles, which the sealing of stock piles is an attempt to retard. A further distinction in the effectiveness of the two grades of oil, lies in their rapidity in "setting," as patches carrying fast, heavy traffic are distorted and roll if too slow in becoming set.

(c) **Methods of Mixing Material.** In the mixing of material for pre-mixed patching, the material is distributed in windrows on a suitable piece of ground or on an abandoned piece of roadway. Oil is applied by a distributor and then mixed by motor graders the same as when operating on the highway itself. In no case, however, is the mixing for patches done immediately upon the the traveled way.

Mixing is occasionally done in concrete mixers. This is a crude and expensive method but it affords a good mix. We have recommended that a number of small portable mixing plants, of say 100 tons daily capacity, equipped with dryers, be secured and assigned one to each engineering district. Their use will allow an earlier start of spring patching, in fact will allow it to begin almost as soon as the roadway is dried off.

During the next year this method may be put into effect. This equipment will much facilitate the drying of materials and also allow the use of the heavier oils and cutbacks.

(d) **Time Stockpiled.** It is not recommended that mixed material be stockpiled for more than two years. We are making a practice of stockpiling when the road is oiled, thus assuring duplicate material for patching. The mixed material is preferably placed on high ground on the high side of the road, peaked in form, and sealed with a road oil or cutback to keep out the moisture. Sealing lengthens the life of the material by delaying the evaporation of the cutback stock. When rapid curing oil is used in the mix, the length of time the material may remain in stock safely is much reduced.

The placing of a sandblanket of at least three inches over the area to be covered by the stockpile aids in protecting it from loss when picked up for use and also carries off moisture.

(e) **Grading and Type of Aggregates.** Idaho attempts to use the same aggregates for patching as are used in the surfacing being patched. When any change is introduced it is toward the use of a slightly leaner mix. In the past there has been a tendency to over-oil the patch because of the oil used for sealing both the base and the finished patch. Oil from these two points added to that of the standard mixed material frequently results in an excess of oil.

For the sake of uniformity in patching sections that have been covered with a coat of rock chips and sand, it is desirable to apply a similar covercoat over the patched area. This has the effect of curing the optical illusion of holes in the road.

The grading of material for patches is the same as that for regular oiling material for bituminous mat which is:

Passing a screen having 1″ square openings	100%
Passing a screen having ¼″ square openings	45-65%
Passing a No. 10 sieve	25-45%
Passing a No. 200 sieve, not over	12%

(f) **Preparation of Break.** The preparation of the margin and area of breaks to be patched is as important as the material applied to the fracture. The edges of the fracture should be cut back vertically on straight rectangular or octagonal lines into the live material about as a dentist prepares a cavity for filling. Wet or damp exposed material in the base should be removed even if it becomes necessary to remove a portion of the unoiled base and replace it with suitable dry base material. At least, a condition should be produced that will permit the patching material to be placed on a dry base for compaction.

Bituminous Surface Maintenance

By

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PROPER DECISION AS TO WHETHER TO PATCH, SEAL, RE-MIX, RETREAD OR REPLACE BITUMINOUS SURFACES.

This decision must of course be made on the basis of known facts pertaining to the particular section of road under consideration.

Patching within reasonable limits of expenditure is necessary and economical, but if the cost is excessive, it may be that the mat is too thin to carry the load in which case adding to the thickness by a surface treatment or a retread might be best if the base is adequate.

If the surface is the slow curing type and the trouble is due to an improperly designed mix or faulty materials, re-mixing and adding whatever is needed will serve the purpose. If the base is weak, it is seldom economical to re-mix or to add to the thickness of the surfacing without reinforcing the base. In the light of recent developments in bituminous base stabilization, it might be better in the case of a slow curing oil mat to scarify and windrow the old bituminous material along the berms and stabilize the base before re-laying the bituminous mat. In some instances it might be better to scarify the old bituminous surface and mix it into the base, adding enough bitumen to make a stabilized base for a new bituminous mat.

SEALING

Practically all of our bituminous surfaced roads are slow curing oil mats and we prefer a light seal consisting of about ¼ gallon of asphalt emulsion or cut-back covered with 15 to 18 pounds of tenth to quarter pebbles or chips. This is an inexpensive seal which we have found to be satisfactory. We use a quick breaking emulsion of high viscosity which does not flow to the sides of the road before the cover can be applied.

Recently, we have been doing some light sealing with about one tenth gallon of MC-2 with a sand cover. The MC material penetrates and bonds itself to the oil mat and forms a tough surface which is not easily damaged by sharp shod horses or ordinary tractor lugs. The cover material is applied with a revolving disc spreader built in our own shop.

SHOULDER MAINTENANCE — LOW COST TYPE

The need for shoulder maintenance on a low cost type of road is usually influenced to some extent by raveling or breaking down of the edges of the bituminous surface. Adequate width of surfacing and a

thickened edge provided during construction by cutting a V shaped channel before spreading the bituminous mix will minimize edge failures. Keeping the berms trimmed down to permit surface water to flow rapidly to the gutters will eliminate a great deal of rutting and some surface failures. This is a maintenance item which is often neglected.

RECONSTRUCTION OF FAILED SURFACES

We are reconstructing a section of road built in 1933 with MC-2 cutback asphalt and graded gravel aggregate. The thickness of the mat was two inches compacted. The subgrade is a silty soil which is difficult to drain and which heaves quite badly in the winter. The gravel base was also inadequate and consequently the surface broke up quite badly.

We excavated the worst frost heave areas and back filled with gravel. We then scarified the old surface and the base to a total depth of six inches and applied one and one half gallons of bituminous material per square yard adding water to facilitate the mixing. On one half the width of the road an SC-3 oil was used and on the other half we used an MC-2, the purpose being to make a comparison of the relative merits of the two materials for this type of base stabilization. In order to mix this small quantity of bitumen with the relatively large amount of aggregate, it was necessary to use about three gallons of water per square yard in the mix.

Power graders were used for mixing, each half of the road width being mixed separately. After the materials were thoroughly mixed and had what seemed to be about the correct moisture content for compaction, the mixture was windrowed on each side of the road and half of the material from each windrow bladed toward the center and leveled off. The spread material was then rolled with heavy trucks, after which the balance of the material was spread and thoroughly rolled with the trucks. The final rolling was done with a seven ton Tandem Roller.

After the job was finished, it was noticed that certain areas remained unstable and cracked at the surface due to excess moisture in the lower part of the base, particularly in the center of the road. This condition is gradually correcting itself as the moisture escapes and it could have been avoided by permitting the lower half of the base to become dryer before spreading the top half or by spreading in thinner layers. It is the intention to apply a light seal in the near future and let it go through the winter before putting on any additional surfacing. It is not advisable to apply a seal coat on stabilization of this type until the mixture is well dried and has built up a resistance to moisture.

Maintenance of Bituminous Surfaces In New Mexico

By

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The type of bituminous surface referred to in discussing the following subjects is an oil treated gravel or crushed rock, prepared by mechanically mixing and spreading the oil and aggregate, either with blades or road machine, the grading of the mineral aggregate conforming to the following specifications:

Material passing through sieve with 1″ square opening	100%
Material passing through 10 mesh sieve	40-60%
Material passing through 200 mesh sieve	5-8%

Grading to be uniform from 0 to 10 mesh

The bituminous binder conforms to the Asphalt Institute specifications for type MC-3 and SC-4.

METHODS OF PATCHING

(a) **Pre-mixed Patching Material vs. Mixed in Place.** Patching of bituminous surfaces in New Mexico is accomplished for the most part with the use of pre-mixed material, provision for which is usually made at the time of construction by providing stock piles of sufficient quantity to take care of the maintenance needs for several years.

Mixed in place patching has been done on some sections of road, but the results as a whole have not been as satisfactory as those obtained with pre-mixed material. A study of the two methods reveals that the principal reason for the difference in results is the greater degree of skill required in making mixed in place patches. A further study of the results accomplished when both types of patches have been properly made does not show any particular difference in quality or coat. Based on these findings, it is believed that for average maintenance conditions, the pre-mixed material will give better results than the mixed in place for surfaces built in accordance with the specification above quoted.

(b) **Type of Bituminous Binder for Pre-mix Patches.** The type of bituminous binder used in New Mexico varies with the geographical location and the length of time which the material will probably be stored. For material which is to be used in the northern or colder sections of the state, and which will be stored for some considerable length of time, it has been found that the type SC-3 oil produces the best results. The asphalt in this oil, having a penetration of 300+, does not become brittle in cold weather, and requires only a minimum amount of handling to produce a workable mix. Material using this type of oil, which has been

stored, does not cake excessively, and for the most part, with the exception of a crust of half inch or so in thickness forming on the outside, remains substantially the same for periods as long as eighteen months.

In the southern or warmer sections of the state, where it is desirable to obtain a harder and more firm patching material, type MC-3 has been used very successfully. About the only objection that may be advanced against the use of this type of bituminous binder for patching material, is the fact that it sets up quite hard within a comparatively short time, which limits its use for stock piled material. For material which will not be stored longer than three to six months, it is satisfactory. For periods longer than this, to avoid excessive waste from caking, oil with a softer asphalt is used.

(c) **Methods of Mixing Patching Material.** The common method of mixing pre-mix patch material in New Mexico is substantially the same process as that for preparing a bituminous surface during construction. The material is windrowed along the side of the road, or nearby, (a section of an abandoned road being used where available), in a quantity which may be easily handled with a motor grader. The material is spread to a width of approximately ten feet the extent of its length, and the proper amount of bituminous binder applied to produce a mixture which will correspond to that contained in the surface, which is to be patched. After the material has been thoroughly mixed, if it is to be stored for any length of time, it is usually placed in piles of, say, 100 to 200 tons, this method of storage permitting only a minimum amount of deterioration from the elements.

For small patches which must be placed during cold weather, small mixing machines of 10 or 12 cubic foot capacity, equipped with heaters, are used.

(d) **Length of Time Material May be Stock Piled.** For average climatic conditions in New Mexico, pre-mixed material may be stock piled for periods up to 18 months without excessive loss of bitumen content, provided the bituminous binder used is properly selected, the New Mexico experience being that SC type oils meet the rquirements better than any other obtainable for materials which are to be stock piled for periods of one year or longer. For materials which are to be stock piled for shorter length of time, type MC oils have been used successfully.

(e) **Grading and Type of Aggregates for Patch Material.** The grading and type of aggregates for patch material, for the most part, conform exactly to that used in the construction of the original surface. There are special cases, however, where it is desirable to reduce the maximum size from one inch to three-quarters of an inch, where the patch to be applied does not extend the full depth of the thickness of the bituminous surface. There are some instances, where the patching amounts to little more than a seal coat, where a finer grade than the above is required. In some instances the size of the material is reduced to give the general appearance of ordinary sand.

SEALING

(a) **Type of Grade of Bituminous Materials Used.** The type of bituminous material used for sealing in New Mexico varies widely, ranging from SC, MC and RC oils to 250 penetration asphalt. On the older surfaces which have dried out, but where excessive cracking has not occurred, excellent results have been obtained by using type SC-5 or type SC-6 oil, with a stone chip covering, ranging between 15 and 18 pounds per square yard, the grading of the cover material being 100% through a ½-inch square opening, and 100% retained on 10 mesh sieve. This same grading for cover material, and in the same amounts per square yard, has been used successfully with 250 penetration asphalt, and with some degree of success with RC-3 cutback. The objection to the use of RC-3 cutback for bituminous binder for sealing is that the asphalt is too hard and becomes brittle during cold weather, permitting the larger particles of stone to become dislodged by traffic wear. Cover material of the above grading and quantity require a minimum of 3/10 gallon per square yard of bituminous material. For new surfaces, and surfaces which are becoming porous to the extent that water will be admitted to the subgrade, type MC-3 oils have been used with a sand covering, sufficient to blot out the excess oil, and have produced results entirely satisfactory.

(b) **Methods of Applying Cover Material.** The most successful method of applying cover material, and the one which has been adopted for general use in New Mexico, is the use of a sand or chip spreader which may be either suspended from the back of a truck or attached to the truck in such a way that the truck may be operated in reverse motion so that the cover material is applied to the fresh surface ahead of any wheels contacting such surface. This arrangement eliminates tracking or marking the freshly applied asphalt, and also affords greater convenience and safety in the operation.

(c) **Methods and Types of Plant Mix Seal Mixture.** The only type of plant mix seal mixture which has been used in New Mexico is natural rock asphalt. This material has been obtained from a deposit located in the state, and has been used upon approximately 300 miles of road. The material in its natural state is sand stone containing about 3% asphalt. Before it can be used for seal coating it is necessary to add approximately 2% asphalt. The grading of the material is very fine and it produces a smooth, rubbery surface which is water proof and which, under favorable conditions, produces a seal coat which will last for several years. This material is usually put down at the rate of 300 to 450 tons per mile for a road surface 20 feet in width, which gives an average thickness of from ½ to ¾ inches. Where natural rock asphalt has been used for sealing material, it has been found necessary to apply a tack coat of RC-1 oil in the amount of 1/10 gallon per square yard.

Maintenance of Bituminous Surfaces

By

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FACTORS AFFECTING DECISION AS TO REPAIR PROCEDURE

There are a number of factors which affect the decision as to the time and particular method or procedure to be followed in performing repairs to failed sections of bituminous surfacing. These factors may be divided into two classes: direct, or those based on an analysis of actual conditions; and indirect, or those based upon the finances and facilities available for making the repair.

Direct factors include engineering investigation of the cause of the failure, past performance record, and maintenance costs of the failed section. Indirect factors would include the relative costs and extent of work involved, organization and equipment available, detours and traffic conveniences, and the general policy of the administrative department.

It is, of course, assumed that most of the states have such a demand for additional surfaced mileage that their decisions as to repair procedure would of necessity be affected by the indirect factors. In any event were conditions ideal as to finances, equipment and organization, major repairs or reconstruction should follow immediately after it became apparent that continuous patching or minor repairing resulted in excessive maintenance cost and such operations were causing noticeable inconvenience to traffic.

The Engineer in charge of maintenance should have an investigation made of the cause of failure before any repairing is made. This investigation should disclose whether or not the failure originated in the subgrade, base or surfacing and also the nature and amount of work involved in placing the surfacing in a satisfactory condition. Practically all failures which occur repeatedly on the same section, sometimes almost immediately after a repair has been made, may be attributed to lack of investigation of the failure or to not making repairs indicated by an investigation. A complete recorded performance history and maintenance cost record of the section involved, together with the information obtained

from the engineering investigation should provide the necessary information concerning the method and procedure for repair.

When the surfacing can be repaired by one of several different methods the relative costs of the different methods should be computed and compared for each section which requires repair. It is often more economical to reprocess a section than it is to place a considerable number of patches if suitable equipment is available. Quite often extensive repairs are postponed due to shortage of funds or because of the large amount of work to be done.

The amount and kind of equipment which can be economically concentrated on one job for a considerable period of time is often a vital factor in deciding when certain types of repairs can be made. If it is impossible or impractical to move equipment away from other maintenance operations the major repairs on a bituminous surface may be postponed until the following year and temporary repairs made to maintain a usable surface. If sufficient equipment were available to perform all major repairs when the occasion demanded, a large part of this equipment might be idle from fall to spring.

Certain types of major repairs of bituminous surfaces cannot be performed economically or safely when it is necessary for traffic to use the road during the repair period. This often makes it necessary to use a different method than the method indicated from the analysis of the failure or the costs involved. Tearing up and relaying of surfacing, reconstruction of the surfacing, sealing or armor coating and most patching can be accomplished with traffic on the road. Base course construction, subgrade shoulder compaction, drainage installations and bituminous subgrade construction cannot be accomplished without considerable added cost for suitable detour or lost motion due to traffic interference, in which case a hazardous condition is created for both traffic and reconstruction crew.

The maintenance policy followed on roads in different sections may vary considerably due to the difference in existing conditions. Some surfaces may provide suitable service by performing only the minimum amount of routine repair and allowing the surface to gradually deteriorate to a condition which requires extensive major repairs or replacements. Surfaces in other sections may be maintained to a high standard by performing major repairs or making replacements at the time the conditions demand. Either policy may be justified by local conditions and finances. This difference in policy makes it difficult to obtain true comparisons of maintenance costs and the proper time for performing certain types of repairs or making improvements.

It is hardly possible to set up formulas or general policies providing a uniform system of repair procedures which would generally be applicable due to the large number of factors involved in the decision. After a thorough study has been made of all the factors referred to in the preceding paragraphs the decision finally made must be based on the expediency and economy and not upon a definite prearranged procedure.

METHODS OF PATCHING

Influence of Conditions

The proper procedure to be followed in placing patches on bitumi-

nous surfaces depends upon several factors among which are the type of surface, kind of base, nature of available patch material, weather conditions and most important the cause of the failure of the surfacing.

Preparation of Patch Location

Regardless of the cause of the failure it is necessary to remove the old surfacing material somewhat beyond the apparent extremities of the failure. A neat outline of the patch should be established. When the surface failure is a failure of the surfacing material itself, it is only necessary to remove and replace with suitable material. When the surface failure has been caused by soft base or subgrade, it is necessary to excavate the wet material and dry it or replace it with suitable material. Mechanical tamping of both surface and subgrade replacement material is very effective.

A surface failure caused by loose material under the surfacing, loosely cemented base or subgrade material or lack of bond between base and surface should be remedied by excavating several inches deeper than the depth of the original surfacing and making a thicker surfacing in the failed section.

Drainage Provision in Clay Subgrades

An investigation of the cause of the failure must be made before any permanent patch or other repairs can be made to a bituminous surface. If the investigation discloses that the base or subgrade or both are unstable when wet it is necessary to provide suitable drainage either of the surface type or blind drains depending upon the type and state of the subgrade soil.

Replacement of Surfacing Material

Before new surfacing material is placed in the patch a good bond should be provided for between the base and the surfacing. Where the base course consists of a stable well bonded material and the surfacing material has the optimum bitumen content a prime or tack coat is not always necessary, but is desirable. A prime or tack coat should always be applied to the base where the surfacing is less than 3 inches in thickness.

The patch should be started by filling in the lowest part first and tamping it so that the patch can then be built up in a uniform layer or layers. In order to obtain a smooth patch it is necessary to thoroughly compact the patch material so that it will form an even surface with the original surfacing material. Additional compaction should be given to the edge of the patch to avoid a possible shifting of the patch material away from the edges of the original surface. Final rolling of the compacted patch is necessary.

When the patch material is apparently quite lean or tends to ravel a seal coat should be placed over the patch and a foot or so beyond the patch soon after completing the patch. The sealing of the portion of surfacing around the patch tends to soften the original surfacing so

that it will smooth out with the patch under traffic and also prevent the possible entrance of moisture to the repaired section. Care should be exercised that excess oil is not used in sealing the patch.

Types of Patch Material

All patch material used on maintenance operations is mixed with blades or traveling mixing plants. Approximately thirty cubic yards of maintenance patch material is taken from the bituminous aggregate windrow and stock piled before the surfacing is laid during construction. This stock piled material is difficult to handle after aging when RC or MC asphaltic oils are used if the bituminous aggregate has been aerated to remove the volatile material. Bituminous aggregate stock piles containing the proper percentage of MC asphaltic oil have been in good condition after 4 years although it is necessary to break up the crust on the outside of the piles. Freshly mixed patch material is easier to use, but is more apt to shove or rut.

Practically no difference has been noticed in the final results obtained where different types of MC asphaltic oils are used in patch material. Stock pile material containing the proper percentage of SC asphaltic oil can be used after seven years if the aggregate is not absorptive. The outside of these piles will show considerable weathering after this time, but the material which is not exposed is still in good condition. Stock pile materials containing absorptive aggregates soon become dry and not workable where SC asphaltic oils were used.

It has been found most economical and satisfactory to patch bituminous surfaces and stabilized base courses with the same type of materials which were used in the original construction. Therefore stock piling of material taken from the road during original construction is very desirable.

General Procedure

Prior to placing the seal coat the surface is thoroughly cleaned by sweeping with a rotary broom. A rotary broom, about 8 feet long, having a reversible sweep mounted on the front end of a wheel type tractor has proven most satisfactory for sweeping the surface. This type permits the use of tractor for other work, such as pulling a roller, disc, drag broom, etc., when not used for sweeping the surface. Being reversible it permits sweeping in either direction and with the wind avoids dust clouds over the surface which creates a traffic hazard, prevents dust from blowing onto the tractor engine and provides better visibility for the operator. The surface application of asphaltic oil is covered with aggregate as soon as possible making the road immediately available for traffic which is carried on one half of the road during the sealing operations.

After the aggregate is placed it is rolled with a smooth steel roller or multiple wheel roller having pneumatic tires. The final operation of dragging with a light broom drag then follows the rolling. Brooming is carried on for several days or as often and long as necessary to keep the surface fairly well covered or until all signs of bleeding have disappeared and the seal has absorbed all the aggregate possible. The amount

of final brooming depends for the most part on the amount of traffic which whips the aggregate to the edges.

Cover Material

Formerly all cover material was stockpiled in small piles located at short intervals along both shoulders. After the application of oil this was hand spread over the surface, however, a uniform coverage was not always obtained by this method. Sand spreaders are now used which are pushed over the freshly oiled surface so that the sand is spread ahead of any equipment passing over the freshly oiled surface. The aggregate is dumped directly into the spreaders and is spread evenly over the surface. This method is more economical and produces more satisfactory results than the hand method. The use of cover material provides a lighter colored surface which is a desirable aid to traffic for night driving and also provides resistance to surface skidding of traffic. The aggregate application generally increases the surfacing thickness from one-fourth to three-eighths of an inch.

Types of Bituminous Material Used

A seal coat of RC-2 or RC-1 is applied at the rate of 0.15 to 0.20 gallon per square yard on surfaces having a high density with smooth tight surfaces. This is covered with about fifteen pounds per square yard of medium sand gravel or armor coat aggregate. An RC-2 or RC-3 armor coat of 0.20 to 0.30 gallon per square yard is applied and covered with approximately twenty-five pounds of armor coat aggregate per square yard on mats containing coarse material or having a rough surface.

MC-2 asphaltic oil is used for sealing on bituminous sand surfaces which were originally constructed with SC-3 asphaltic oil and which have a satisfactory stability. Generally 0.20 gallon per square yard is applied but this quantity must be varied with the season of the year. The cover material used on sealing of bituminous sand surfaces is blow sand which is usually obtained from the roadside. The cover material is not applied until after the penetration of the asphaltic oil is about completed and is then applied in excess of the amount required.

On bituminous sand surfaces originally constructed with cutback asphalts either MC-2 or RC-2 can be used as seal material. These surfaces are usually very hard on the surface and subject to considerable cracking due to the lower three inches of the mat remaining soft. The principal reason for sealing these surfaces is to prevent the entrance of sand into the cracks which forms a layer of separation between the upper ½ to ¾ inch of surface and remainder of the mat. Entrance of surface moisture into these cracks has caused no apparent damage during a period of five years.

RECONSTRUCTION OF BITUMINOUS SURFACES

The reworking of bituminous mats is divided into two groups: those which have had little or no special maintenance, and those which have had considerable special maintenance and are again in need of special treatment.

Bituminous mats which have had little or no special maintenance and which have developed considerable wrinkling or subgrade displacement are torn up, remixed, the base allowed to dry, and then relaid if the bituminous aggregate appears to contain sufficient asphaltic binder to allow the mat to compact. If the mat appears deficient in bitumen, samples are taken at 1,000 to 1,500 foot intervals, analyzed, and compared to the original test data for the particular section. RC-2 asphaltic oil is added to the material found deficient in binder for mats originally constructed with SC asphaltic oil. For mats where MC asphaltic oil was originally used, MC oil is added in maintenance operations. If the mat is relaid without the addition of oil, it is generally sealed immediately.

Mats which have had considerable special maintenance and which again need special maintenance are reconstructed in a different manner after the decision has been made as to the time to make the repair. The mat is torn up, removed from the surface and stockpiled or it is windrowed on the shoulders, and the subgrade stabilized. This stabilization is based on the subgrade samples taken prior to the original construction or on samples taken after tearing up the mat. Generally a stabilized gravel mat is constructed using new materials. In most cases it has been found more economical and more satisfactory not to use binder or other material from the subgrade in the construction of stabilized gravel bases which are to be constructed on expansive types of subgrade soil. The stabilized gravel base is constructed to a thickness of three to five inches on types A-6 or A-7 soils depending on the height of the fill and the drainage conditions. The depth of stabilization is based on engineering judgment rather than on any engineering formula. Bituminous stabilization is resorted to on sandy and silty soils which are economically suited for that purpose. The depth of this stabilization depends on the quantity of the material in the subgrade soil that passes the 200-mesh sieve. A three-inch depth is generally sufficient where the subgrade soil contains from twenty to forty per cent of material passing the 200-mesh sieve, a four to five-inch depth is required when the per cent of material passing the 200-mesh sieve is between ten and twenty per cent. Stabilized gravel bases after compaction are primed with MC-1 asphaltic oil. Bituminous stabilized bases are not primed. The gradation of the bituminous mat is then rectified by the addition of the proper materials and the asphaltic oil content is corrected by the addition of the proper asphaltic oil.

Some of the early mats constructed were only one and one-fourth to one and one-half inches in thickness and when these mats are torn up approximately one-fourth inch of the base material is incorporated with the mat. This material, together with about one-fourth inch of material added by seal coating, increases the thickness of the reworked mat about one-half inch. After a mat has been torn up a second time, the addition of gravel is necessary to correct the gradation. This also increases the thickness of the mat.

The strengths of reworked mats are usually much higher than for mats originally constructed with SC asphaltic oil. This is due to the use of RC asphaltic oil in the reconstruction which fluxes with the SC asphaltic oil and produces a heavier residual asphalt in the mat.

It has been found necessary on several bituminous mats to seal surface cracks which were caused by shrinkage of the subgrade. This shrinkage is probably due to an unusually severe winter or a loss of moisture in the subgrade. These cracks occur chiefly on mats where the stability of the bituminous aggregate was very high and the bitumen content low.

The reworking of bituminous sand surfaces is materially different from the reworking of bituminous mats due to the nature of the subgrade material and the greater thickness of the combined base and surfacing.

Bituminous sand surfaces which are deficient in asphaltic binder are reworked to a depth of two to three inches and after adding the proper quantity and type of asphaltic oil are remixed. Reworking to a depth of less than two inches is an invitation to early failure due to slippage of the top layer on the construction layer below caused by lack of sufficient depth of uniform material to transmit the shearing action of the loaded area.

Bituminous sand surfaces which are low in stability and which have been built on a dry sand subgrade and contain little or no material passing the 200-mesh sieve must have their stability increased to give better load distribution. This involves complete reconstruction and usually the addition of filler material.

Bituminous sand surfaces which have the proper asphalt content, stability and density, and which fail due to displacement of the subgrade may be reworked to restore the proper cross section as a temporary maintenance operation. If the failure was caused by the loss of moisture in the sand subgrade and the moisture content returns to normal (between three and five per cent) this type of maintenance is effective.

TABLE I

TOTAL MILEAGE OF BITUMINOUS SURFACES AND THE APPOXIMATE MILEAGE GIVEN SPECIAL MAINTENANCE TO JANUARY 1, 1938

Year	TOTAL MILEAGE OF BITUMINOUS SURFACES						TOTAL MILEAGE GIVEN MAINTENANCE EACH YEAR							BITUMINOUS SAND SURFACES					
	Total Mileage Bit. Mat Surfaces 1¼-3" Depth	Bituminous Sand Surfaces 4-6" Depth	Bituminous Sand Mat Surfaces 3" Depth	Surface Treatment On Stabilized Base	Surface Treatment On Existing Road	Bit. Mats Constructed on Stabilized Base	Sealed or Light Armor Coated Const. Funds	Sealed or Light Armor Coated Maint. Funds	Torn up & Relaid Due to Subgrade Failure Const. Funds	Torn up & Relaid Due to Subgrade Failure Maint. Funds	Reconstructed Due to Mat Failure Const. Funds	Reconstructed Due to Mat Failure Maint. Funds	New Stabilized Base Course Constructed	Sealed Const. Funds	Sealed Maint. Funds	Torn up & Relaid Due to Subgrade Failure Const. Funds	Torn up & Relaid Due to Subgrade Failure Maint. Funds	Reconstructed Due to Surfacing Failure Const. Funds	Reconstructed Due to Surfacing Failure Maint. Funds
1928	5									1									
1929	5	5																	
1930	5	22																	
1931	10	82																	
1932	116	150						30		1				7		2			13
1933**	151	157						48		15	5	15		11		7			16
1934	362	157						31		8		3		32		9		6	5
1935	449	168	21			15	93	78	1	41	4	4	4	17		4		0	9
1936	598	206	57	21		138	112	74	0	17	0	0	6	16		5			12
1937	738	216	91	221	11	248	24												
Total	738	216	91	221	11	248	229	261	1	83	9	22	10	83		27		6	55

Grand Total Miles of bituminous surface including 1937 construction not completed—1,277

Summary of Total Mileage Given Special Maintenance

Total Bituminous Mat Sealed .....490 miles of a total of 738 miles
Total Bituminous Sand Sealed ..... 83 miles of a total of 307 miles
Total Bituminous Mat Torn Up and Relaid ..... 84 miles of a total of 738 miles
Total Bituminous Sand Torn Up and Relaid ..... 27 miles of a total of 307 miles
Total Bituminous Mat Reconstructed ..... 31 miles of a total of 738 miles
Total Bituminous Sand Reconstructed ..... 61 miles of a total of 307 miles

Grand Total mileage of bituminous surfaces, torn and relaid or reconstructed 203
Grand Total Special Maintenance .....776 miles of a total of 1,277 miles
Grand Total Special Maintenance less Const. Sealing ....547 miles of a total of 1,277 miles

** Includes maintenance in previous years.

Some Phases of Maintenance of Bituminous Surfaces in the Province of Alberta, Canada

By

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Before discussing the subject proper, it should be stated that the Province of Alberta is more or less in the initial stage of Bituminous Surfacing, so that it's Maintenance organization for Bituminous Surfaces is in the development stage. Necessary changes are being made from time to time to keep pace with each successive year's construction.

The District Engineer has charge of Maintenance in his District, and supervises and controls the work of small section crews, who take care of minor repairs, such as patching, and all hand work. For more extensive repairs and maintenance, such as sealing, retread, or replacement of surfaces, special crews are organized, and work under the supervision of a junior or Resident Engineer who has had some experience in Bituminous work.

The Province of Alberta has only two types of Bituminous Surfaces to maintain, Plant Mix, consisting of a mat of 2 to 2½ inches in thickness, of densely graded aggregate, mixed with a slow curing liquid asphalt, which in this country goes under the designation of P.M.H., a liquid asphalt very similar to the SC-5 material. There is only some 78 miles of this type of Bituminous Surface in Alberta.

The other type of surface, of which there is now a little over 270 miles in existance, consists of light penetration treatments composed of mats varying in thickness from ¾ to 1 inch. The aggregate in these thin mats run from densely graded to medium coarse materials. The binder used in these mats is composed chiefly of medium curing liquid asphalts of the MC3 and 4 type, though some slow curing materials have been used.

In order to maintain these surfaces and arrive at a proper decision as to whether to patch, seal, remix, etc., we have no organization as you have in some of the states, such as a division of tests, an organization which by a proper analysis and study of the cases can help you to arrive at the true causes of the failure. However, it is absolutely essential that a thorough study of the situation be made.

In this Province the District Engineer reports on the failures to the Highway Commissioner, a study is made of the road by the Highway Commissioner, the District Engineer, and possibly a Resident Engineer who has had experience in Bituminous work. A program of work is then agreed upon and submitted to the Minister and Deputy Minister for approval.

We know of no rule of thumb to guide us in arriving at a proper decision as to what should be done. However each case is analysed and viewed in the light of proper engineering practice and experience. Each group of failures is studied separately in an endeavor to arrive at the correct cause of failure, for if this can be done, we can arrive with some degree of certainty at the proper remedy, and thus decide whether to patch, seal, remix, or replace the surface. This decision may have to be modified, due to lack of maintenance funds, lack of sufficient and proper equipment, or to the fact that the volume of work required to be done in a short season is too great to allow for sufficient time to be spent on the particular job. In such cases a more or less temporary measure has to be resorted to.

Roughly speaking, we in Alberta have to deal with two classes of failures, (1) Failures of the Bituminous mat, (2) Failures of the base or subgrade.

Failures under class (1) may be caused by faulty design; improperly graded aggregate, the wrong type of asphaltic oil, too rich or too lean a mix. Other causes of the failure of the mat may be due to lack of aeration, moisture within or between the mat and base. Disintegration may have been caused by cattle, horses, narrow tired wagons, lugs on caterpiller or wheel tractors.

Failures under class (2), that is failures in the subgrade or base, which have been transmitted through and caused failures in the mat, need more attention, and are costly and difficult to rectify. Failures in this class include frost boils, soft spots, change in the density of the base, causing shrinkage or swelling, poor drainage, little or no base support for the mat, etc.

Once the true cause and extent of the failures have been arrived at, it is a matter of rectifying, as far as funds and equipment will allow, the causes of failures, and restoring the road as nearly as possible to it's original condition. Bearing these points in mind, we are more or less guided into the proper decision as to whether we should patch, seal, remix, retread, or replace the damaged Bituminous surface.

METHODS OF PATCHING

(a) In Alberta pre-mixed patching material on the heavier mats has been found to be far more satisfactory than mixed in place material. In the first place you have far better control over the proportions of liquid asphalt to the aggregate, and consequently you get a mix that is neither too lean nor too rich, as is often the case with mix in place material. Secondly, it has been found that new and inexperienced patching

crews get on to their jobs more quickly, and do better work than when they have to resort to mix in place patching.

For light surface treatments, the mix in place method is generally used, though this depends to some degree on the extent of the patching rquired. For patching small segregated breaks, the pre-mix material has proved more satisfactory, for the same reasons as those given above for the heavier mats.

(b) In Alberta different types of Bituminous binder for various patches have not been considered to any extent. The practice has been to use as far as possible the same type of binder for patching and repairing as has been used in the original mat. This of course is not always feasible. In connection with the plant mixed mats referred to previously, after the stock piles that had been made during construction had been used up, the binder used for subsequent patching was the same as that employed in light surfacing treatments. The binder most extensively used for all patching purposes in Alberta is the MC type of asphalt. Some RC materials have been employed with satisfaction, but we have not had very good results from early mats where SC material was used as binder. For this reason SC asphalt is not used for patching. This year in some sections we are again trying out SC asphalts, but it is still too early to report on the results.

(c) Methods of mixing pre-mixed patch materials vary. Our work is not extensive enough to warrant the purchase of mixing machinery, so that a good deal of the work is done by hand, mixing on a platform and in some cases drying and heating the aggregate in a very similar manner to mixing small batches of concrete. For more extensive work, and stock piling, a concrete mixer is used. This has proved to be quite efficient and satisfactory. For small penetration patch work asphalt kettles, with motor driven spray attachments, are used. On more extensive work, pressure distributors and power maintainers are used for mixing and smoothing out the aggregate.

(d) We have very little information or experience with regard to the length of time the material may be left in stock piles, as we do not make a practice of stock piling the material ahead for any length of time. The material is very seldom left for more than a day or two in stock piles, before it is used. We have found however that stock piles made with M.I.P. asphalt, can be left for a week or ten days without any ill effects, except for a thin crust covering the outside of the pile, caused by weathering. This M.I.P. asphalt as we term it, is an MC type, but has a softer base than the general run of MC materials. I might also mention here that stock piles made with the P.M.H. asphalt mentioned previously in connection with the plant mix mat, were left for two years, and used for patching. However, by the end of the second year on cool days picks and crowbars had to be resorted to, in order to loosen up the pile.

(e) With regard to the grading and type of aggregate used for patch material, we in Alberta follow the practice of using the same type of aggregate as has been used in the original mat. When the gravel is being crushed for the construction of the mat, provision is made to

crush and stock pile extra material for maintenance purposes. Should these stock piles be used up rapidly, it is an indication that extensive repairs are necessary. A crusher is then brought back to the pit, and crushing is carried out for seal coat, retread, replacement, etc., as the case may be, and again stock piles are left for maintenance.

The present trend in Alberta is to get away from the densely graded type of aggregate for all grades of bituminous surfacing, as the following extract from the specifications show:

(1) AGGREGATE FOR ROAD MIX—

Passing a 1½ inch circular opening	100%
Passing a 1 inch circular opening	60-95%
Passing a ½ inch cricular opening	25-50%
Passing a ¼ inch circular opening	0-15%
Retained on a ⅛ inch circular opening	100%

(2) AGGREGATE FOR SURFACE TREATMENT—

(a)

Passing a 1 inch circular opening	100%
Passing a ¾ inch circular opening	90-100%
Passing a ⅜ inch circular opening	20-50%
Passing a ¼ inch circular opening	0-10%
Retained on a ⅛ inch circular opening	100%

(b)

Passing a ¾ inch circular opening	100%
Passing a ½ inch circular opening	80-90%
Passing a ¼ inch circular opening	5-25%
Retained on a ⅛ inch circular opening	100%

(3) AGGREGATE FOR SEAL COAT—

Passing a ½ inch circular opening	100%
Passing a ⅜ inch circular opening	70-80%
Passing a ¼ inch circular opening	10-30%
Retained on a ⅛ inch circular opening	100%

In conclusion I might say the Province of Alberta is endeavoring to build and maintain, as far as her limited funds will allow, roads that will enable the public to travel in safety. In doing this, she is fortunate in having neighboring states to the south, who have passed through the early stages of highway development, and from whose experience and knowledge Alberta may now benefit.

Maintenance Organization and Methods Used in Patching Bituminous Surface in Montana

By

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MAINTENANCE ORGANIZATION

All Maintenance Activities in Montana are supervised by the Maintenance Engineer, stationed at the headquarters office, in Helena. He supervises the expenditures of all the funds allotted by the State Highway Engineer for Maintenance purposes. He advises and assists the Division Engineers in handling all the Maintenance work in their respective divisions.

The highway system of Montana is divided into ten divisions, with a Division Engineer in charge of each, who is responsible for all Maintenance and Construction work.

In each division the Division Engineer is assisted by a Maintenance Foreman or a Resident Engineer. Regular routine Maintenance is supervised by these men, who in turn are assisted by sectionmen.

The Sectionmen handle all regular routine Maintenance over their definite section. The mileage covered by each sectionman is from twenty-five to seventy-five miles, the length of the section is determined by the type of Maintenance required. With one or two regular assistants the Sectionmen handle all the regular Maintenance work such as: cleaning ditches, shoulders, remixing short sections, cleaning culverts, burning weeds, painting guard rails, patching the oiled surface and removing snow.

The headquarters for state highway equipment is located in Helena and it is operated under the direction of the Equipment Engineer. The Equipment Engineer is assisted by a Field Equipment Superintendent, who travels throughout the State. He inspects all equipment frequently and reports his findings to the Equipment Engineer and Division Engineer.

All major overhaul and repair work is done at the headquarters shop. The regular repair work is done in the division shop, which is located at the division headquarters. Each division shop employs from two to four machinists.

During the spring and summer months special crews are sent throughout the state, from the central headquarters in Helena, to do different types of special work. This Maintenance work consists of: center line striping, crushing and placing gravel, seal coat oiling over large sections, cleaning slides and ditches and remixing large sections of bituminous treated material.

While in the various divisions the Foreman of each crew reports to the Division Engineer under whose supervision the work progresses. The work done by these special crews is usually of superior quality as they are well equipped, well organized and highly specialized.

METHODS USED IN PATCHING BITUMINOUS SURFACES

(a) Pre-mixed patching material versus mixed in place on heavy mats and light surface treatments.

Most of the patching on Montana Bituminous surfaces is done with pre-mixed material. The only time patches are mixed in place is on mats that cannot be torn up and remixed. Montana has very few sections of roadway with mats of this type. However, this procedure was carried out on a rough surface of a 95 Road Oil Mat during the past season.

On the light surface treatments regular top course gravel is used with a heavy cutback to patch the failure. If the roadway in question is badly pitted or broken up, the mat is torn up, oil added and a light mat made to replace the surface treatment.

(b) Type of bituminous binder for various patches.

Prior to 1934 most of the pre-mixed material used an SC-2 or 3 road oil as the bituminous binder. From 1934 to 1937 many MC cutbacks were used, but the stockpiles with this material tend to dry out making it very unsatisfactory for patching.

The tendency at the present time is to return to the SC Road Oils in pre-mixed patching material. In all patches where pre-mixed material is used, RC-1 cutback is used to prime the hole.

All sections where patches are mixed in place SC-3 Road Oil is used, as the subgrade conditions at these points are usually unfavorable and the mat has to be torn up from year to year.

Upon failure of surface treatments MC-3 cutback has been used with gravel, except when the mat is torn up, then SC-3 Road Oil is used as the bituminous binder for the light mat.

(c) Methods of mixing patch material.

Most of the pre-mixed material used is mixed beside the stockpiles of gravel with a motor patrol. Oil is delivered in trucks from the refinery to the mixing place, where it is transferred to a distributor. It is then distributed into the windrow of gravel where it can be readily mixed with a motor patrol.

If material is needed for patching in the spring or fall when the

mineral aggregate in the stockpiles are cold and wet, each division has a small pugmill equipped with a drier and a mixing unit. Approximately twenty-five yards of suitable patching material can be mixed with this unit in a working day. This mixing plant is easily transferred about the division as required. However, the cost of mixing with this machine is much higher than with the motor patrol, so it is only used when absolutely necessary.

All the patches mixed in place are treated in the same manner as a light road mix treatment, except the old mat is tacked with RC-1 just before the patch is ready to be spread over the old surface.

(d) Length of time material may be stockpiled.

In Montana we have some pre-mixed material, mixed in 1930 using an SC-2 Road Oil as the bituminous binder and we find this material is still very suitable for patching.

In 1934 and 1935 some gravel was mixed using an MC-2 cutback as the bituminous binder, but this material tended to dry out and in some cases additional cutback had to be added prior to using it on the roadways.

The length of time material may be stockpiled depends largely upon the type of bituminous binder used.

(e) Grading and type of aggregates for patch material.

The following is the gradation table of aggregates used for patching in 1933.

Gradation Table

Size	Per Cent
Passing a screen having 1¼-inch circular openings or 1-inch mesh openings	100%
Passing a screen having ¼-inch circular openings not more than	65%
Passing a 10-mesh sieve	30-55%
Passing a 200-mesh sieve	5-15%

This material did not prove satisfactory as there was too much large aggregate in the mix which made it difficult to make a thin patch.

The following is the gradation table of the aggregates used almost exclusively now, and which has proven very satisfactory.

Gradation Table

Size	Per Cent
Passing a ¾-inch screen (circular)	100%
Passing a ¼-inch screen (circular)	40-65%
Passing a 10-mesh sieve	30-55%
Passing a 40-mesh sieve	15-35%
Passing a 200-mesh sieve	5-15%

The above data explains the methods of patching used in Montana to maintain an extensive mileage of bituminous surfaces on limited revenue.

Maintenance of Bituminous Surfaces

By

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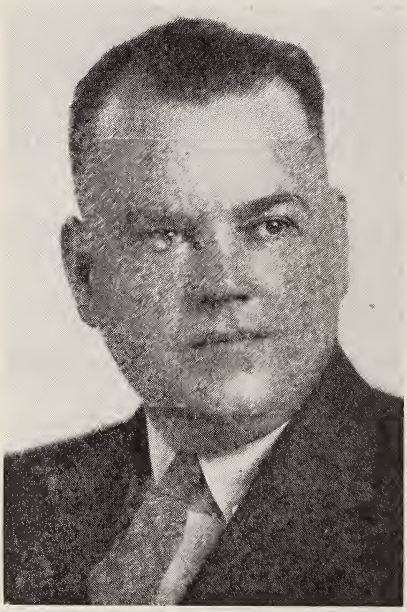
PROPER DECISION AS TO WHETHER TO PATCH, SEAL, REMIX, RETREAD, OR REPLACE BITUMINOUS SURFACES.



We have all been confronted with the necessity of making the above decision. If this decision was arrived at only from the engineering standpoint it would be comparatively simple, however, the question of finances is ever before us. In thinking of this question the following questions must be settled:

(1) The amount of money the particular road earns and its earning capacity compared to its desired condition and general appearance.

(2) Available finances.

(3) Condition of base.

(4) Surface and percentage needing repairs.

Unfortunately entirely too often we are prone to sacrifice road appearance with consequent adverse public reaction in favor of saving a few dollars per mile.

We must first satisfy ourselves as to the true condition of the base. If the base is obviously a failure there is little justification for spending an additional sum of money on a new surface or even repairing the old surface. The only answer to this is reconditioning the entire base. If conditions justify it we may, of course, improve the old base by incorporating into it the old top. It might be of interest here to state that recent experiments in Texas indicate an advantage in using a soil stabilizer type emulsion in very small quantities in helping to set-up base. This emulsion is used in lieu of a part of water. This has been done in new base construction and we see no reason why it would not be advantageous in old base in reconditioning old base material. It is not the idea here to arrive at a stabilized base as we consider it, but rather to aid setting-up.

Remix. Remix of old top presents in some cases a very intricate problem and I am afraid sometimes our desire to experiment overcomes our better judgment. We may go as far as to say that any type of

material can be remixed but all types certainly cannot be remixed economically. There are definitely a number of types of pavement which under certain conditions should be remixed, such as surfaces constructed of soft asphalt—SC types which are rolling and shoving due to excess binder, (surfaces which have become water-logged due to capillary action and are, therefore, rolling and shoving.) This particular type of failure is characteristic of light materials with no base. Of these types particularly adapted to remixing we would say in general that surfaces which show failure of more than 25% of the total area and where base repairs are deemed necessary over a large area—these should be remixed.

Retread. Where the present material shows evidence of being brittle and cracking due to lean mix and where the base condition is good and where a seal coat would not be sufficient to bring the surface to the desired thickness necessary to carry traffic, we would retread. We would retread also where the surface has remained in good condition over a long period of time but where subgrade settlement has caused an irregular riding surface.

Sealing: As stated above under remixing we would seal when approximately 25% of the surface needs it. Patching is a year to year complaint and if as much as 25% of a road surface needs repairs we may safely assume that the following year we will have additional patching. A complete job of light sealing is comparatively inexpensive and should furnish a good riding surface for from five to eight years with very light, if any, maintenance cost during this period and, of course, a much better public reaction. The question—when to seal—is very important and should be brought to the attention of engineers at every opportunity. Repair crews should not be advised by the engineer to "repair where it needs it" but maintenance should be watched as carefully and as intelligently as construction to save the initial investment in the road.

Replace: We would say that when to replace a bituminous road comes under the same question as to when to remix. Roads not subject to remixing must be replaced when the base is obviously a failure and needs reconditioning throughout.

Patch: We should carefully consider the relative cost of yearly maintenance patching as against sealing; as well as cost we should consider public reaction. In general a much better appearing road will result from a road which has been sealed over a road that has been spot patched.

METHODS OF PATCHING

(a) Pre-mixed Patching Material Versus Mixed in Place on Heavy Mats and Light Surface Treatments.

The question of pre-mixed material versus hot patching is one that I have heard heatedly discussed. Pre-mixed material undoubtedly has an advantage from the viewpoint of safety to the public during repairs. Premix material is easier to handle and if the patch is of sufficient size results in a smooth and well appearing patch. However, if the patch is small in area, much may be said in favor of the hot patch. Hot patching requires a man of more experience on the road but does not require a

central plant. A patch of pre-mix material must be relatively open in texture to facilitate curing—hot patches can be made much more dense. We should, of course, always bear in mind that the patching material should be wherever possible identical to the material that the road was originally built of.

(b) Type of Bituminous Binder for Various Patches.

The type of bituminous binder to be used should be based on the penetration of the asphalt originally used in the pavement. In general probably a slightly harder asphalt should be used because in the patch we have a type of penetration macadam. If the type is pre-mix, an RC cut-back would be suitable in most cases; if hot-mix, regular paving cement.

(c) Methods of Mixing Patch Material.

Methods of mixing patch material usually depends on the present preference of the operator, based, of course, on his choice of type of patch. Of course, if the repairs are small the material may be mixed by hand.

(d) Length of Time Material May be Stockpiled.

Oklahoma has used stockpiled material of the SC type after more than a year. Practically any pre-mix material of the cut-back or emulsion type may be safely stockpiled for thirty days. If the aggregate is relatively soft we will find that these soft particles have absorbed a considerable amount of the volatile material. This, of course, will hasten setting-up. Stockpiles should be built relatively high and when taking material from the stockpile we should be careful to make clean cuts and not disturb more of the surface than is necessary.

The length of time material may be stockpiled is, of course, greatly dependent upon temperature and humidity. The curing-out of cut-back materials is definitely a function of the ability of the atmosphere to take up moisture. High temperature is not necessarily conducive to curing-out since with high temperature often goes high humidity.

(e) Grading and type of aggregates for patch material.

If pre-mix material is used we must use a relatively open mix since we must hold the road open to traffic. A considerable amount of volatile material must be eliminated or the patch will be shoved off the road by traffic. In general I suggest about 30-35% plus 10 material with maximum size 3/8".

Local or creek bank deposits are being used successfully but every effort should be made to use the same aggregate in patching as was used in the original construction. Gravel aggregate should certainly be used to repair a road where the original construction used gravel aggregate. Any additional cost in securing a blending patch is certainly offset by the final appearance.

SEALING

(a) Type and grade of bituminous materials used.

We must first consider priming the base. If the base surface is sufficiently well bonded a prime might be unnecessary. If the surface is relatively loose a prime is necessary—a medium curing cut-back will

penetrate the base to a depth of about an inch and will deposit its asphaltic content principally in the upper layer. Slow curing oils are not recommended because of the tendency to develop a spongy mat.

RC materials are probably the most popular of the cutbacks because they are readily reduced to its basic asphalt. In penetration we find about equal preference between 135 and 230 penetration materials.

(b) Methods of applying cover material.

Spreader boxes are used in most cases. On applying by hand material should be deposited lengthwise of the road.

Reconstruction of Failed Bituminous Surfaces

By

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In repairing failed bituminous surfaces the most important consideration is promptness. The old saying "A stitch in time saves nine" certainly applies to bituminous repair work. Any breaks in the surface of a bituminous road should be repaired within twenty-four hours. This may sound a bit radical, but radical practice is necessary in bituminous road maintenance. Thousands of dollars of maintenance money can be saved by the prompt repair of surface breaks. Prompt repair of surface breaks precludes the maintenance of bituminous surface by the gang method which makes only periodic repairs. The patrol method of maintenance is much better adapted to bituminous road maintenance.

Next in importance to prompt repair of surface defects is the careful determination of the cause of the failure and the complete curing (if possible) of the defect. It might be argued that these two theories are opposed to each other, that it is impossible to make prompt repairs and at the same time find the cause of the failure and cure such cause. This brings out the difference between permanent and temporary repairs.

It is true that frequently time does not permit the thorough examination and study necessary to determine the cause of failure or possible weather conditions interfer with the complete elimination of the cause. In such cases temporary repairs should be made immediately. Such temporary repairs should waterproof the failure and put the road in shape till more complete and lasting repairs can be made. So-called skin patching adapts itself to this type of repairs very nicely. This is done by applying about one-fourth gallon of bitumen per square yard to the failed surface and covering the bitumen with twenty or thirty pounds of aggregate. If the failure is fairly deep two or three such applications may be made one on top of the other.

After temporary repairs have been made or even before any work is done (if time permits) a careful study should be made to determine the cause of the failure. This study will insure the making of adequate repairs and will also enable the engineer to guard against future mistakes of the same sort.

Most surface failures are due to the failure of the base. This may be due to improper base construction or as is frequently the case to the overloading of the base. More attention should be be given to the limiting of loads on black top roads so that they will not be called upon to carry excessive traffic. At least they should not be expected to carry loads in access of the maximum permitted under state laws.

Maintenance of Bituminous Surfaces

By

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PROPER DECISION AS TO WHETHER TO PATCH, SEAL, REMIX, RETREAD OR REPLACE BITUMINOUS SURFACES.

Failures on bituminous surfaces generally are of the same type regardless of the type of surface and can be roughly classified as base failures or surface defects or failures. A large number of the latter type are often incorrectly diagnosed as such, when, in fact, they are only the result of inadequate base support. While it is true that failures in all types can be classified alike, maintenance and repair methods are by no means similar.

In order to be able to make the proper decision as to the type of repair the cause of the failure should first be correctly determined. This requires a thorough knowledge of all the conditions which may have been contributory factors to the cause of the failure. Climatic conditions and seasonal variations in type of traffic may be the influencing factors in certain cases. After the type of failure and the cause of such failure has been determined, the proper decision as to whether to patch, seal, remix, or resurface can readily be made. One factor which is frequently of vital importance in considering repair methods is whether the alignment and grades are to be changed materially in the near future. When such changes are to be made, it is often uneconomical to make extensive repairs or improvements as it is usually difficult and expensive to salvage any of the existing base or surface. In such cases temporary repairs of a less expensive nature can often be made which will adequately and satisfactorily carry traffic for a short time. It is often difficult to determine just how extensive repairs are to be made by the patching method. On account of the large amount of hand work involved, this is probably the most expensive type of repair encountered on bituminous surfaces. The season at which repairs are made will influence to a large extent the type of repair to be made. It is needless to say, however, that patching should be done whenever holes develop in the surface regardless of the time of the year. If small breaks are allowed to remain, disintegration of both the base and surface will soon progress to such a point that much more extensive

repairs will have to be made. Patching can in most cases be considered only as a temporary method of repair either as a preparation for some other type of repair or to serve until some other type of repair can be made. In some cases where the surface is supported by an adequate base course and only a few minor surface or base defects show up, patching may be the most economical method of repair and will, if such patches are properly constructed, give satisfactory service for a long time.

Patching requires skill and should be very carefully done. A good patch should in a short time blend with the adjoining surface so that it will become unnoticeable. Patches should be made of the same type and grade of material as used in the original construction, and be placed so that the density will be as near like the adjoining pavement as possible. If these precautions are not taken the patch will in most cases turn out to be either a high spot or a low spot.

It is an established fact that bituminous materials harden more or less under the action of the elements. This hardening process is more rapid in open mixtures which contain a large number of air voids. As a result of this hardening process of the binding medium the surface course soon becomes brittle and the surface will crack under very small movements of the supporting base or subgrade. This cracking will further expose the surface to the action of the elements and traffic, and as a result disintegration and raveling will be materially accelerated. All of the popular types of low cost surfaces should preferably be sealed as soon as possible after construction so as to close up the air voids and keep surface water from entering the surface course or base. Such seal coats should consist of sufficient bituminous material, together with a covering of clean, durable and properly graded aggregate, to properly close air voids and provide a satisfactory nonskid surface. Light seals such as "penetration" or "fog seals" which do not include the use of cover material are of doubtful value for this purpose and if repeated often enough or in sufficient amounts will often result in a slippery surface. Penetration seals can often be used to add binder to a bituminous mat which has been built with insufficient oil. In choosing this type of seal to be used one factor which should not be overlooked is, namely, the safety factor which in relation to the surface is the resistance to skidding.

Bituminous surfaces of the oil mat type which have become dry and brittle to such an extent that raveling is taking place can be readily salvaged by scarifying and pulverizing and adding the proper amount of binder and remixing.

This method is not, however, any cure for a surface which has failed due to inadequate base support and its success or failure is dependent on the correct diagnosis as to the cause of the failure; it is also a relatively expensive form of maintenance. Surface failures such as rolling or shoving are caused by either an excess of oil in the mat or the presence of too much moisture. The first opinion that most trouble of this type was due to excess oil has been more or less superseded by the belief that excess oil alone sometimes causes corrugations, but usually moisture is the direct cause. Where the mat contains an excess of oil it can be remixed with additional aggregate and relaid. Excess moisture can be

eliminated by remixing; however, no permanent results can be obtained by this method if the aggregate is hydrophilic in character.

It is usually more difficult to determine when resurfacing is desirable. This method of repair depends somewhat on the type of surface which is to be resurfaced and the results desired. Minor irregularities and settlements can be eliminated by resurfacing with thin courses properly placed. The load carrying capacity of such surfaces is also increased by successive resurfacings. Before resurfacing to provide increased load carrying capacity, all conditions should first be carefully considered as the thickness of the base can usually be increased at less cost per inch of thickness than the cost of providing such additional thickness by the resurfacing or retreading method. Thin bituminous surfaces can sometimes be greatly improved by resurfacing. When resurfacing to improve the riding qualities as many of the irregularities as possible should be taken out by means of a wedge course in order that the final wearing course may be placed to as near a uniform thickness as possible. This is important in order that uniform compaction may be obtained during construction and that any further compaction under traffic will be uniform.

When failures are of such nature that none of the above maintenance methods are deemed adequate or advisable in order to place the bituminous surface in good condition, replacement of the wearing course will, no doubt, be the proper procedure. This is hardly a maintenance operation except where only short sections or small portions of a section are to be replaced. The causes of failure should first be carefully searched out and any failures due to poor drainage of either surface or subgrade drainage should be corrected; usually the base failure is caused by water, often held back by a rock ledge or impervious soil layer in the subgrade, or by inadequate side drainage. Whatever the cause, it should be searched out and remedied before replacing the base course. A bleeder drain should be placed through each shoulder so that subdrainage will be provided.

Increasing loads may require heavier construction in order that the road may properly serve the traffic. Where surfaces are of the low cost type they can often be incorporated in the base along with other aggregates or bituminous materials, and thus utilized to add thickness and load carrying capacity to the base before a new surface is placed.

Each job will usually present different possibilities due to local conditions and will therefore have to be considered on its own merits. This is especially true of low cost bituminous surfaces which are mainly dependent on their low cost to the availability and use of local materials.

No hard and fast rule can be laid down as to when to use any of the various methods of repair since no two sets of conditions may be of a like nature. As stated previously in this paper, the method will depend largely upon judgment, experience and a thorough knowledge of local conditions.

SEALING

In Missouri this type of work can be roughly divided into two

general classes; that is, surface treatments which demand the use of a light grade of bituminous material without the use of, or with only negligible amounts of cover material, and those which utilize the heavier grades of bituminous materials and must have a substantial covering of aggregate to prevent traffic from picking up the bituminous material.

The first type mentioned calls for bituminous material which will flow readily at ordinary summer air temperatures. This is necessary in order that the material may flow into the pores of the mat to as great a depth as possible. Cutbacks of both the rapid curing and of the medium curing type are being used for this type of seal. When this type of seal is placed under traffic, the amount placed in any one application must necessarily be small in order not to inconvenience traffic any more than necessary. Sometimes a light covering of chips may be necessary to keep traffic from picking up the bituminous material. This has a tendency to reduce the effectiveness of the seal as the cover material will take up some of the bitumen. This type of seal can be very effective in raising the bitumen content of lean mats if placed on new work before traffic has had an opportunity to fully compact the mat. One disadvantage of this type of surface treatment is that it must usually be performed every year, another is that successive applications have a tendency to make the surface slippery in wet weather which, from a safety standpoint, is a highly undesirable condition.

Cover material should not be applied on this type of surface treatment unless necessitated by traffic or in case the surface voids are large. When used to partially close up surface voids the cover material should be small enough and in such amounts that the surface voids will not be completely filled.

The second type of surface treatment which requires the use of a heavier type of bituminous material can be made to serve two purposes: The first is waterproofing mats, and the second is to provide a nonskid surface. This type of surface treatment requires the use of heavy cutbacks or high penetration asphalts, which will hold a considerable amount of cover material. Lighter grades of bituminous materials can be used if the cover material is to be mixed with the binder before rolling. Where heavy binders are used by the inverted penetration method a light colored surface can usually be obtained, which, if properly placed, may be effective for a considerable length of time. Light colored surfaces are desirable in that they promote safety for night driving. Surface treatments of this type do not penetrate the mat being surfaced but rather form a thick film over the surface and are therefore an effective means of keeping out surface water.

Cover materials which are composed of crushed angular particles are desirable as they will bond better with the bituminous material than smooth, rounded particles. The maximum size of cover material shall be governed by the quantity of bituminous material to be used. For applications at the rate of approximately one-fourth of a gallon per square yard cover material should preferably not be over three-eighths of an inch in maximum size. For applications of from four-tenths to five-tenths of a gallon per square yard cover material up to one-half or

five-eighths inch in maximum size may be used. When cover material used for surface treatments has too much spread in gradation, a considerable amount of the larger particles may be whipped off by traffic, thus resulting in a loss of this material. Some spread in gradation is desirable in order that the particles will be keyed together as much as possible. In order to overcome the loss of cover material, and to make the surface treatment more dense, recent developments are toward the application of cover material in two sizes; the larger size being applied to the bituminous material first, thus allowing them to bond to the surface. The smaller particles being applied last will sift down into the voids between the larger particles and tend to key them in place.

The quantity of cover material will vary with the conditions under which the surface treatment is placed as well as with the type and amount of bituminous material used. For best results cover material should be dried and heated before being applied.

Before the surface treatment is placed the existing surface should be thoroughly cleaned of all foreign materials and should be thoroughly dry in order that a good bond will be insured. When this work is done under traffic, applications should be made to only one-half of the road at the time.

The application of cover material should follow immediately behind the distributor and each distributor load of bituminous material should be covered before the next one is poured. Cover material should be placed by means of mechanical spreaders of such types that the amount of material will be uniformly applied in the desired quantity. If a light colored surface is desired a slight excess of cover material should be used. This is necessary in order to prevent traffic from bringing the binder up and tracking up the surface before the binder has cured out or hardened. The surface should be rolled once over with a light roller, either tandem or three-wheel, weighing over five (5) tons, shortly after the cover material has been placed. If the cover material is damp, rolling should be deferred until the cover material is dry. It may sometimes be advisable to broom the surface lightly to assist in the proper distribution of the cover material and cover up thin spots.

Cover material may also be placed directly from tail gates from trucks or may be spread by hand with shovels. Neither of these methods is very accurate, and an excess is usually placed. The cost of a good spreader will soon be offset by the savings in the cover material when the specified amount can be accurately spread.

The subject of plant mix seal mixtures is in the experimental stage in Missouri. We have placed two miles of an Amesite mixture on an old oil mat road and it has been in use one year and looks very fine. The seal varies from regular 2″ of Amesite surface over several of the ⅛ mile sections to only the ¾″ wearing course over others of the ⅛ mile sections.

The distress and failure cracks in the old pavement do not show now, and we are expecting this experimental section to solve the sealing of some of our oil roads with the higher type asphalt mixtures.

RECONSTRUCTION OF FAILED SURFACES

Failure of low type bituminous surfaces are of two kinds: failure in the surface and failure of the base or sub-base. The majority of failures have been due to the base or subgrade; those due to the surface itself are few and more easily corrected. The failures due to the surface are caused by a dry mat or an insufficient amount of bituminous binder, one with an excess of bituminous binder, an excess of fines in the aggregate, and due to an insufficient amount of fine material in the aggregate.

A dry mat or one constructed with an insufficient amount of bitumnous binder can best be corrected by removing the surface and remix with additional bitumen. Sometimes the dry mats have been protected and the usefulness prolonged by penetrating the surface with additional bitumen since the depth of penetration is not very great; this is at the most a cheap, temporary repair to a dry mat.

A mat constructed with an excess of bituminous material will result in bleeding, which is easily taken care of by blotting with fine aggregates, the gradation of the aggregate depending on the amount of bleeding.

A mat constructed with an excess of fines in the aggregate may result in shoving and corrugations in the surface if an excess of oil is used. To correct this, coarser material must be added which probably would require the remixing of the surface material.

A mat constructed with an insufficient amount of fines in the aggregate will result in an open mat subject to surface ravel and the admission of surface water. This condition can be corrected by the addition of bitumen and fines to close the voids in the surface.

These failures in the bituminous surface are a very small percentage of the failures since the control of the aggregates and bituminous material are watched so closely that little difficulty along these lines is experienced in late years.

The failures due to subgrade distortion or shifting of base is by far the greatest cause of failure in low type bituminous surfaces. These failures are due to the presence of moisture in the roadbed, this moisture is either surface or underground water. Failures in the surface that have been caused from surface water are the more easily repaired and protected from a repetition. If the base has been badly distorted it should be restored to its original cross section by scarifying and blading and the addition of aggregate to the base as needed and the mat replaced and a seal coat applied to the surface. The shoulder should be made as nearly waterproof as possible and the side ditches kept open and clean to facilitate a rapid run-off.

Failures in the subgrade or base of the road due to underground water are the greatest source of trouble both in numbers and in difficulty to correct. If the source of the water can be found and trapped it is an easy matter to drain it away from the roadbed. If the source of the water cannot be found and the subsoil is so heavy that it will not free the water,

it is then either necessary to replace the subgrade material with material that will free its moisture or build up a base thick enough to bridge the unstable soil in the subgrade. For the most part in this state this condition has occured in small areas and we have found it cheaper to build up a heavy base and bridge the unstable soil rather than to undertake its replacement.

SHOULDER MAINTENANCE OF LOW TYPE BITUMINOUS SURFACES

(a) There has been some discussion in the past as to whether the shoulder of the road should be built up after a low type bituminous surface had been placed and in this state a number of years ago several sections were left as constructed without building up the shoulder for a year. This did not seem to reduce the edge failures and the appearance of the section was not as good as those sections that were shouldered. The edge of the mat was exposed and required considerable work to take care of the edge ravel. Since then all low type surfaces have been shouldered up the same as the high type pavements. The shoulders on low type pavements are maintained the same as on higher type pavements. The ruts are eliminated by dragging, particular attention being given during the winter and spring months, and every effort is made to keep the water drained away. When the frost is leaving the ground in the spring the maintenance men roll the shoulders with their 36-inch width patch rollers. These are pulled behind their trucks and will give a pressure of 120 to 130 pounds per inch of width. This does a very good job of compaction and prevents the moisture from softening the shoulders to a very great depth. A shoulder slope of at least ½″ per foot should be maintained.

Our maintenance men in this state agree that intensive shoulder work in the winter and spring months reduces the amount of moisture that might reach the edge of the mats and pays large dividends during the remainder of the year.

(b) The question of sealing low type bituminous surfaces has been a subject of much discussion in this state and it is my belief that in the great majority of cases a seal is economically justified. Where a surface is dry and subject to raveling or open enough to admit surface water, it surely needs a seal to protect it and the seal will easily pay for itself in a few years. If the base of the road is subject to ground water and wet weather seeps, the sealing will tend to hold this moisture and in time will cause a bad base failure. This is about the only condition where a seal would not be justified. Until one is sure the water is trapped and draining permanently from the base of the road, a seal would be wasted.

(c) In order to protect the edge of bituminous surfaces the width of roadway primed and surfaced has been increased and with each increase in width of treated surface maintenance work along the edge has decreased. By increasing the width of the prime, surface moisture has been held away from the edge of the mat. The entire width of the shoulder has been oiled in some instances and the effect on the treated surface has been quite noticeable. The results obtained from shoulder oiling has justified the cost and this type of work will undoubtedly be

increased in the future. The construction of the bituminous surface the full width of the roadbed would create a substantial saving in maintenance and justify the additional cost in time. In most cases a surface width of twenty-two feet with the shoulder water-proofed by oiling or an armor coat, is justified easily by the saving in maintenance

SECTION V

Round Table Discussion

Round Table Discussion

CHAIRMAN, F. V. REAGEL, Engineer of Materials
State Highway Department
Jefferson City, Missouri

Mr. Reagel: Gentlemen, as Mr. McKinnon said, this is a round table discussion to pick up the loose ends and to ask any questions that we have been wanting to ask that we haven't had time for, or that overlapped in the program. I have heard some statements that maybe some of our scientists were shooting a little over our heads. Maybe this is a point where we can discuss the A-B-C's as well as the X-Y-Z's.

I have a few questions that have been handed to me by the various chairmen, of points that they do not believe have been discussed sufficiently, or that they did not have time for. I will bring up some of those, and then we will ask for subjects from the floor.

The question is asked, "Where Proctor compaction is used, what technic is in use to determine the density being obtained in the field?"

Apparently someone desires to have a brief description of the actual technic in use. I think that would be very well in order. Do we have someone who would like to volunteer to give that information?

I. E. Russell: Wyoming

I imagine there are others here who had a great deal more experience with soil compaction than we have. For the past two years we have been watering and rolling our grades. Our procedure consists of taking preliminary soil samples along the right-of-way and making laboratory determinations of optimum moisture and optimum density. Results of these tests are returned to the field engineer to serve as a guide as to the amount of moisture and rolling necessary to obtain desired densities. We attempt to get from 90 to 100 per cent of optimum density on actual construction. During actual construction density tests are frequently made as the fill is brought up to grade. This is done in the conventional manner by digging a hole in the grade, weighing the soil removed, determining the contained moisture and correcting for same. Volume of hole is, of course, determined by sand of known weight per cubic foot.

The samples of soil removed in making field density determinations are forwarded to the laboratory. Here optimum density determinations are again made and checked against those obtained in the field so as to be sure sufficient density is actually being obtained on construction. In that way we feel that we are approaching the solution of this problem as nearly as we know how.

I am vitally interested in this subject myself, and I will be glad to have someone here tell us more about it.

N. W. McLeod: Saskatchewan

To what extent is the Proctor needle used in getting the results of this compaction?

Mr. Reagel:

Is anyone using the Proctor needle determination? We don't use it extensively.

J. H. Swanberg: Minnesota

We have been using it in getting the maximum compaction. As Mr. Russell suggests, we have made determinations in conjunction with our Proctor determinations. It isn't a routine procedure; we have just used it on at least two jobs. We have correlated insofar as possible the results we have obtained in the field and the results we have obtained by compacting that material at optimum moisture in the laboratory.

Mr. Reagel:

In this connection it might be stated that discussion yesterday brought out that the limits in use in general seemed to vary from 95 to 100 per cent in the specifications.

Has anyone else anything to offer in this connection?

I. E. Russell: Wyoming

I got off on the wrong foot yesterday. I thought they were talking about this subject and they really were

talking about bases. Perhaps some of you remember that I said that when we had flat curves develop in the laboratory we didn't attempt to go to the full 100% on our compaction in the field, because that would often times take a good deal more water than we felt we could afford to put on the job. Strange as it may seem, we are paying as high as $3.00 per thousand for water, and when you jump from a natural water content of 6% and to around 20 or 22% in order to get a few more pounds, it doesn't seem the economical thing to do.

I would like to have an expression of opinion on that from some of you.

Mr. Reagel: As I understand your question, Mr. Russell, it is whether it would be desirable to go to the full 100% at an extra cost of water, or would it be better to take the more economical action and be satisfied with 95% compaction where water is expensive. Does anyone have an opinion?

J. H. Swanberg: Minnesota The only thought that I would have in that connection is that we have found in some laboratory studies following the compaction of soils, and I assume you are speaking of soil compaction, where we have compacted those samples at optimum moisture content, optimum density, that upon subjecting the samples to alternate freezing and thawing in the laboratory we have reduced materially the density of the soil samples. If that is experienced in the field, there might be some question as to whether you need to go to the full 100% compaction.

Freezing and thawing apparently have some effect in causing bulking of the material.

Mr. Reagel: I might say in this connection that the Bureau of Public Roads, in connection with Mr. Hogentogler's work, have classified soils into, I believe, at least three groups, one group of soils which would contain those with which no particular trouble would be anticipated. They would permit perhaps as low as 90% compaction. Then they would have a second group, poorer quality soils, that would require 95%; and still a third group that they would want to require 100% on, and they separate the groups by a weight per cubic foot factor. If anyone is interested, he might check on that. I believe there might be some logic and value to using a differential percentage, depending upon the soil that is found.

W. H. Wood: Texas I want to ask Mr. Russell what the conventional method is for measuring the volume of the hole from which the field sample was taken. Do you dry the sample in the field and get the dry weight?

I. E. Russell: Wyoming Yes.

W. H. Wood: Texas — In connection with the Proctor needle, I want to call attention to the fact that when you run into solid material, if you hit a rock with it, it will blow up.

Mr. Reagel: — I might say, in connection with the use of sand for determining densities, that there is a conventional little piece of equipment that consists of a fruit jar with a screw top to which has been fixed an inverted funnel, with a stop-cock closure, and this fruit jar can be taken to the field with prepared sand and you simply invert the funnel over the hole and turn the stop-cock, which gives a conventional method for performing that test.

Here is a companion question: "How high can the plasticity index of a soil be, and still get satisfactory results from Proctor compaction?" Would anyone care to venture a maximum limit? That is a definite question. Certainly someone has a limit beyond which he would not care to go. Does anyone have a specification limit for that feature? Do you have that, Mr. Powers?

J. W. Powers: Arizona — It seems to me that the particular question could only be answered in one way, and that would depend upon the percentage of material minus 200. You might have a comparatively high plastic index and have a very limited amount of minus 200 material and be able to compact it very readily, but as you began to run your 200 up, say around 70, 80 or 90%, I doubt very much whether it could be done. I have in mind soils in our state that I doubt could be compacted to ultimate density, especially those that run from 75 to 90% minus 200. I don't believe the moisture content that would be necessary for maximum density could be obtained in the field without excessive cost, and I doubt very much whether you could use equipment over the material if you did have it, because of the excessive slipperiness that would develop. I have in mind a plastic index in the neighborhood of 35 to 50.

Mr. Reagel: — That is a very good answer. Does anyone else care to discuss that point?

We seem to have a series of Proctor questions here. The next one: "Is the Proctor compaction ever used for cuts?"

I might say in that connection that our department in Missouri has for the past two years, at least, I believe, made a practice of using Proctor compaction in cuts, on the theory that it is better to have a uniform condition of subgrade through a rock cut than to go from one type of support to another and perhaps to vary even in the cut in the degree of support.

Has anyone else had any experience with that?

J. W. Powers: Arizona

This isn't in answer to the question, but it is a question that arises in my mind in connection with compaction. Is there any particular piece of equipment that seems to be better for one type of material, or that can be generally used, rather than any other type of equipment, for the purpose of obtaining maximum density, whether it be a tandem roller, a three-wheeled roller, a rubber-tired roller, or any other type of compaction equipment?

Mr. Reagel:

Mr. Powers, do you refer to fill compaction, in which you probably would desire to use the sheep's foot roller?

J. W. Powers: Arizona

That is the question I am asking, whether the sheep's foot roller is better than possibly the rubber-tired roller or a flat-wheel roller. The sheep's foot roller has gained a lot of popularity because it does do a good job of compaction. However, in our state we find that in sandy and gravelly types of material the sheep's foot roller is a good one to use only up to a certain point, because it imbeds itself in the material and keeps pulling up the rock and it is necessary to finish compaction with some other type of roller equipment.

We find in our state that the best compaction we can get is by the use of the rubber-tired roller in sandy and gravelly types of material.

Mr. Reagel:

We have made quite extensive use of the sheep's foot roller and I have also used the multiple-tired pneumatic type of roller. I believe we find that the sheep's foot roller would do the initial compaction, starting from the bottom upward, better than the multiple-tired roller, but that during the later stages perhaps the rubber-tired roller would do just as well.

I noticed at the last Asphalt Institute meeting the question of using the flat sheep's foot roller on sandy materials was brought out, I believe by Mr. Weathers in his paper, and his opinion coincided with yours. He brought out the fact that in connection with sandy material it was rather useless to attempt to use a sheep's foot, and that some other type of roller would be necessary.

O. L. Stokstad: Michigan

On one project in our state we used a roller consisting of a series of cast iron discs that served very well in the same capacity as a sheep's foot would. It had the advantage of leaving a smoother surface than a sheep's foot. I don't know that it would be quite as good as a sheep's foot in compacting the lower portion of the layer to be compacted, but it did a very nice job and the boys seemed to like it better than a sheep's foot.

Mr. Reagel:

There is this further to be said about a sheep's foot roller, I think. We have noticed in connection with some

of our work on finishing compaction with the sheep's foot roller that the marks of the sheep's foot roller persist at the surface, and under certain conditions there is a tendency toward lamination at that point, and an attempt has been made to overcome that by putting a light harrow or drag of some sort behind the roller in the latter part of the last stage of the rolling. I believe there is some merit in doing something of that kind.

Roland Vokac: New Jersey — I would like to ask Mr. Stokstad if he could give us the approximate dimensions of the disc roller he speaks of.

O. L. Stokstad: Michigan — I don't have the dimensions nor the name of the concern manufacturing the roller. If Professor Housel is here, he may be able to give you that information.

Roland Vokac: New Jersey — Just approximate dimensions. What do you have on the diameter of the discs, how spaced, how thick, and so on?

O. L. Stokstad: Michigan — From a mental picture of the roller, it seems the discs were about three feet in diameter, with a bearing area about an inch wide and a spacing between the discs of about four inches.

I. E. Russell: Wyoming — We have done some work in our state with field rollers.

V. G. Gould: Iowa — You were talking about a disc roller. It happens to be the Parsons roller, manufactured in our state. We have used it on one job. The discs are about forty inches in diameter and about two inches wide. I am just making a guess on that; I don't know.

In regard to the sheep's foot roller, I think that the reason we are getting so far apart in our opinions is that it depends on whether you are working at the critical (or the optimum) moisture how much value you get out of the sheep's foot as compared with the other type of rollers. If you are trying to compact almost any of these materials, excluding the sandy soil mentioned by Mr. Powers, and trying to compact within a range which is ordinarily around 1.5% of variation, it seems to me that the sheep's foot, through observation, is nearer foolproof than any other type of roller.

Also, due to the fact that particularly on batch-mixed base material, where your necessary moisture content is generally about 3% above your optimum and you need to dry back to the high optimum before you start compaction, if you used the other type of rollers you would require very close control of your moisture content or your compaction effort would be lost. The sheep's foot will come nearer detecting the optimum moisture of its own accord than any other type of roller. Also with the other type of rollers, if the moisture content is excessive, the surface is sealed and the excess moisture can-

not escape, and if the moisture content is below optimum, compaction is difficult to attain. So, from my observation to date, it would seem that for general use probably the sheep's foot has a good many advantages.

The disc roller that you were speaking about comes in the same category from that standpoint as your other rollers, because it likewise is sealing in the excess moisture. Otherwise it is a good roller. If you are working within a narrow range of optimum moisture, it seems the sheep's foot has less chance to get you into trouble than any other type.

Mr. Reagel: The sheep's foot roller is made with a number of types of shoes or barefoot, or however you want to express it. Has anyone any experience with using a variety of sizes of shoes or using the roller without the shoes, and if so, have they any recommendations to make to the rest of us as to which is preferable for this type of work? The protrusions that tend to get smaller at the ends, or those that are square, or those that have definite shoes on the ends—does anyone have any opinions in connection with their use?

J. A. Johnson: Missouri In the last issue of the American Road Builders' Association proceedings there is an article by Hogentogler, of the Bureau of Public Roads, showing varying densities at varying moisture contents with varying compaction pressures indicating that varying densities could be obtained by increasing the weight of the sheep's foot tampers.

Mr. Reagel: Did he make a comparison between different types of feet on the sheep's foot roller?

J. A. Johnson: Missouri Not as I recall, but I believe practically all the rollers made now have about the same size feet, about seven square inches.

V. G. Gould: Iowa Perhaps I may be able to shed just a little light on that point. Last winter at the soils school in Washington, D. C. it was brought out that there is no method of pre-determining the weight per square inch required for the sheep's foot roller to secure the Proctor density upon walking out. Of course the ideal condition is to use a pressure per square inch which will assure the desired density when the roller has walked out. However, there is no way in the world to determine whether you want the 10.5 square inch foot or the 7.5 square inch foot, or whether you need smaller or larger feet, if you want to get a specified compaction when you have walked out.

With regard to the indentations left on the surface, we have had pretty good luck so far, (I am not saying we won't come to grief before we are through), by stopping the rolling with the feet about an inch from the

surface, mulching that, and then using the pneumatic and steel roller for the final rolling. I won't presume to say this method will work all the time, but it has been successful so far.

Mr. Reagel: Does anyone else have anything to offer in that connection?

J. W. Powers: Arizona

In that connection, speaking of the possible densification of materials, there was an article that appeared in Engineering News Record some time in the past, I don't remember when, in which they intimated (and probably had data to support) that you got more compaction from the vibration of the Caterpillar pulling the roller than you did from the sheep's foot roller itself. That may be the reason you can't pre-determine the densification you are going to expect by the weight of the roller. I don't know whether there is anybody who has ever studied that. That isn't my thought; it is merely information that has been developed from, I presume, somebody's experience.

Mr. Reagel: If there is no further discussion in that connection, we have this question: "In view of Professor Housel's suggestion that the mat fails before the subgrade bearing capacity is developed, are well-graded mats, uncemented, to be preferred for thin mats to those which are well cemented by clay and Portland cement?"

It simmers down to, "Do you prefer uncemented mats in the case of thin mats, or tightly bound mats?" Does anyone have an opinion on that?

H. G. Nevitt: Missouri

I might continue my discussion of the first day when I talked on this subject. I see Professor Housel just came in, and he can talk on it also. I think a lot of this question of the proper thickness and type of base is tied up with the type of mat. I assume in this question that where he talks about an uncemented mat for a thin mat he means an uncemented base course under a thin bituminous mat. That is what I would take from the question.

It seems to me that a great deal of the differences of opinion the other day came about in this way: For example, we had the Iowa people commenting that they had thin mats on stabilized base courses and that they were quite successful, didn't have any cracking or evidence of over-design. I think I know why. They have a very well balanced design. It is truly a stabilized base with a balance between the various components, which has a relatively high strength.

Out of the Rocky Mountain States, as Mr. Powers describes his conditions, he has a natural soil mat. It isn't stabilized, intentionally. The natural gradation of

the base course is fairly good and you get excellent bearing power, but probably per inch of thickness you have less bearing strength. You probably need for the same conditions, which must always be kept in mind, a little thicker mat of that kind. Therefore you might have a more rigid base there; whereas, in Iowa they might have a fairly flexible base, excellent in load carrying ability, and therefore the thin mat is all that is needed on top of it to give a total bearing structure which is both flexible and strong. Under some other conditions we might have to go to very thick base courses if we had thin mats, and the total resultant structure would be very rigid and would not utilize the available strength of the subgrade.

I think, in considering thin versus thick foundation courses and thin versus thick mats, the type of both the mat and the foundation course should be kept in mind.

Mr. Reagel: It occurs to me, when the gentleman includes Portland cement in his question with regard to thin bases, that certainly we wouldn't want to take a very thin mat using Portland cement as a binder. I am thinking of the possibility of salvaging that mat in case it was too thin. The way I would think of that is that if we are going to design a rigid slab base, we had better design it to start with, with an ample factor of safety, because if it goes to pieces on us it is going to be something else again when we try to do something with it, whereas with the more flexible type, if it is too thin it simply might be a matter of concern only temporarily, because we can come along and add thickness.

I don't know whether that gets at the meat of the question in mind or not.

N. W. McLeod: Saskatchewan On Tuesday, Professor Housel stated that stabilized or consolidated gravel base courses may have to carry in shear, the whole of an applied wheel load, because their rigidity is frequently such that the small amount of deflection causing their failure is not sufficient to develop the bearing capacity of the subgrade beneath.

The subgrade bearing capacity is frequently, possibly usually, of the same order of magnitude as the punching shear of the base course, and is usually worth while developing in the interest of base course economy.

This would appear to be a strong argument in favor of the densification of particularly the upper layers of the soil subgrade by Proctor technique, etc., in order to reduce the amount of deflection required to fully develop the subgrade bearing capacity.

It is possible that Mr. Stoddard's reported success with rigid stabilized gravel bases in Iowa may be due to such densification of the subgrade, intentional or otherwise, with the result that the base course and subgrade are behaving as a unit in developing reaction to the applied load.

Where such subgrade densification is not practically possible, as Professor Housel pointed out on Tuesday, either the base course must be constructed sufficiently thick to carry the whole of the applied wheel load in punching shear, or sufficient flexibility must be introduced into the base course itself so that the deflection of the base course at failure, or preferably before a failure, is sufficient to develop the full bearing capacity of the subgrade in its natural condition.

In connection with the rigidity of base courses, there are some points I would very much appreciate having Professor Housel discuss.

There would seem to be the possibility when a well-cemented clay stabilized gravel base fails due to punching shear, that it would be divided into a layer of unjoined blocks, which would have at least some tendency to transmit the full applied wheel load to the subgrade practically undiminished, just as an equivalent layer of loose bricks might. On the other hand, when a consolidated gravel base fails, there could still be a very considerable lateral distribution of load in line with Professor Housel's arch theory of the stability of granular materials.

Take the case of two base courses of the same thickness over the same subgrade, one base course of clay stabilized gravel, the other of consolidated gravel, each base course being too rigid to develop before its failure, any subgrade reaction from the given subgrade. Now consider the result of applying a wheel load just sufficient to cause failure of each base course. The consolidated gravel base upon failure comes into contact with the subgrade, and under the applied load will develop the full subgrade reaction. Is there not the possibility that the new combination of developed subgrade bearing capacity plus the punching shear, which remains in a consolidated gravel base due to arching action even after so-called failure occurs, may provide more than sufficient reaction to the applied load for equilibrium conditions, even though this was not possible for the original rigid consolidated gravel base? This would seem particularly possible where some kind of bituminous wearing course existed to provide a confining influence. It would seem that this might not be

possible for the clay stabilized base, where failure into blocks might transmit the load undiminished to the subgrade.

Also which do you consider to be the more flexible of these two types of base for any given thickness, and quantitatively, how much deflection would you expect each of these bases to show before failure for say a five or six-inch base course and a bearing area twelve inches in diameter? I would very much appreciate having Professor Housel discuss these points if he would kindly do so.

Prof. W. S. Housel: Michigan

Mr. McLeod brings up several interesting points in his discussion which serve to emphasize important considerations in the success of low-cost road surfaces. In the first place, the densification of the soil subgrade appears to be particularly promising and, incidentally, is a possibility which has been generally overlooked in the past. Load tests conducted by the Michigan State Highway Department on a so-called flexible surface have indicated a marked deviation in the rigidity of the road surface as compared to the rigidity of the subgrade. If this is generally true it may be necessary to increase the rigidity of the subgrade in order to effect any considerable economy in the thickness of the stabilized base and surface. As Mr. McLeod has stated, if it is not possible to produce the subgrade rigidity comparable to that of the superimposed surface it may be necessary to provide a surface thick enough to carry the entire applied load in punching shear, more or less discounting subgrade bearing capacity as an effective part of the supporting capacity.

The other alternative is to incorporate proper degree of flexibility in the road surface itself so that it may deflect in such a way as to develop the subgrade bearing capacity before or at least in conjunction with the distributing action of the surface. Mr. McLeod's remarks in this respect deal with two possibilities, a consolidated gravel base with very little if any cohesion, and the stabilized gravel base to which binder is added to furnish cohesion. It is possible that a consolidated gravel which allows some degree of movement of the individual particles may be the more flexible of the two types. It appears that this type of flexibility would have to be rather carefully controlled because if the relative movement of the individual particles was sufficient to result in some rearrangement of the particles, the probability would be that the base course would gradually become less and less stable as time goes on.

In the case of the stabilized base, while it may break up into blocks, the relative movement of these

blocks would provide some degree of flexibility and at the same time retain some interlocking along the fractured faces which would be capable of developing shear providing the blocks were not displaced horizontally so as to destroy the interlock. It is possible that this type of base would be the more permanently satisfactory. It seems to me that the most desirable type of surface to perform the function of distributing the load and at the same time have considerable flexibility, would be that in which the binder were semi-fluid material such as bitumen having a considerable amount of ductility in itself.

This brings up a point that has been mentioned in other discussions regarding the importance of ductility as a property of bituminous binding materials. There has been a tendency to discount ductility as an important property in this connection. It seems to me that flexibility which appears to be an important and desirable feature of low-cost surfaces may be to a certain extent a function of ductility of the binding medium used.

In summing up my discussion of the question which Mr. McLeod has brought up, I would say that it is difficult to determine which of the two types of base he mentioned would be the more flexible and at the same time durable. In order to determine this it will be necessary to have much more experimental data on the load carrying capacity of such surfaces and the deflections of these surfaces under load. Without any more information than is available at the present time I believe that I would favor the stabilized gravel base which broke up into blocks but may still retain some interlocking across the fractured faces over the consolidated gravel base which it appears to me might deteriorate much more rapidly. It is my present feeling that the most promising method of regulating the relative rigidity of road surface and subgrade is that first suggested by Mr. McLeod of densification of the subgrade soil as the first step in construction of the low cost road surface.

J. W. Powers: Arizona

I think I have a case in point there which will more or less illustrate what happened to us. We ultilized in Arizona, particularly in the northern areas, a material we call cinders, which is a volcanic material with very low P. I. and comparatively low binding power. We placed six inches of this material on one of the roads which had a subgrade that was probably A-7 type with somewhere in the neighborhood of 50% to 70% minus 200. The six inches that were placed on there would not stay in place at all. In other words, the flexure of the base when it became wet caused a disturbance of the internal structure of this material, and all you

could do was keep blading it off and, as it broke, keep blading it, until finally you got down practically to the subgrade.

It was our opinion at the time that if you increased the base thickness to somewhere in the neighborhood of twelve or fourteen inches it would stay in place and you would not get the movement underneath which continually broke the bond. So it was tried, and since they have put this twelve or fourteen inches on there (it was supposed originally to be twelve but it ranges from twelve to as high as fifteen inches) that material has stayed in the roadway and has made a very satisfactory road without breaking up, so that you have probably done what Professor Housel said. You have increased the rigidity of your base and you are not transmitting the load to the subgrade, and as a consequence it stays in place.

We based our opinion that it would do that upon the fact that there was a change in subgrade type within a comparatively short distance of where it was so very bad, and that particular material was disintegrated sandstone which has comparatively low P. I. and comparatively high breaking value, and when it was originally put on the only piece that was good in the road was where you had the disintegrated sandstone underlying it.

Mr. Reagel: Does anyone else care to add to this subject?

I don't want to direct this discussion by questions to the point of excluding any questions you folks have.

I have another question: "What are the various states using in the way of soil classification independent of the Bureau of Public Roads classification of eight groups?"

That comes into the discussion that was had here the other day, the point, as I get it, being whether any of us are using a different classification.

O. L. Stokstad: Michigan uses a system of soil classification which differs from the groupings as developed by the Bureau of Public Roads.

Soil surveys were started in 1925 before the eight groups had been established, and at that time the only classification available was the one developed in the science of pedology. The pedological system was, therefore, adapted to highway requirements and the success of the adaptation warrants continued use of the system.

Mr. Reagel: I would say that in Missouri we have just decided to utilize a soil survey in connection with each of our projects, and we have been making some comparisons with

the work of other states to arrive at just what is the best procedure of working up that information, and we have concluded to try to use some more descriptive terms for our soils than just the numerical designations, with the thought that perhaps we can get our engineers familiar with the materials a little better than if we just used a numbered classification.

Are there any other offers on that?

Another question: "What is the place of positive and negative spot bituminous material in bituminous mats?"

I don't know just what the intent of that question is. I assume that it is an effort to find out whether both positive and negative spot materials are being used, and whether or not they are being used satisfactorily. I might say that so far as we are concerned, we use positive spot materials practically entirely in the case of SC mats. We differentiate between the two materials when we start using a material that we desire to act as a positive binder, such as the cutbacks. We start requiring a negative spot with the medium and rapid curing cutbacks. With the slow curing cutbacks where we don't expect to procure a positive bond from a residue we permit the use of positive spot materials, and since they are more economical in price we get practically all of the materials negative spot in the SC grades.

Does anyone wish to discuss that?

If no one cares to discuss that question, the next question is, "What has been the experience of various states concerning the use of powdered asphalt and flux oil?"

J. H. Swanberg: Minnesota

I might say we have one project about six or eight miles long which we laid about three or four years ago, a plant mix using a flux oil with powdered asphalt. The road has been, I would say, very satisfactory, but the cost of it is higher because of the greater percentage of bituminous material used than if we had used either tar or cutback. The sum of the oil and powdered asphalt is possibly a per cent higher than we would normally use in using an MC-4. The results have been quite satisfactory.

W. J. Slavin:: Michigan

I might say that we have used powdered asphalt in patching mixtures, mixed with SC material. The results so far have been very good. I felt for some time the desirability of having a little more asphalt in an SC patching mixture in order to give a little more stability to the patch after it is laid and rolled down. We use

about one per cent of powdered asphalt in the mix. That is an uncracked SC.

Mr. Reagel: We have SC mats, using cracked materials, in which we have incorporated varying percentages of powder ranging from ten to twenty or twenty-five per cent powder, and we believe we find a benefit from the use of from 10 to 15 per cent powder. I believe we get our money's worth from that incorporation.

We find no additional benefit from 20 per cent as against 15 per cent, or some such percentage.

I also believe it is desirable to pay some attention to the compatibility of the flux and the powder, if you are going to use powder. Some materials would not satisfactorily combine.

The next question is: "When failures occur in bituminous mats which cannot be blamed on the subgrade, what factors are chiefly responsible for the failures?" That is a good one.

J. W. Powers: Arizona It seems to me that that question is a rather broad one, but it would depend in many cases upon what would be considered a failure. In other words, you would have to define the word "failure" before you could answer the question.

Mr. Reagel: We might simmer that down to, "Assuming we do not have subgrade failures, what are our major troubles with mats?"

H. G. Nevitt: Missouri I thought that was a pretty good question, because sometimes I don't think the troubles are analyzed, and just to sort of start the ball rolling I would say there are two major troubles. First, lean mixes. I believe that is probably the biggest trouble nowadays, the tendency to go to such lean mixes which later on ravel or cause trouble. We have had trouble in the past with rich mixes, so the tendency has swung back. I don't mean by that that I am opposed to staying below the tolerance, but I do believe that is a more common trouble than almost anything else.

The other trouble is water trouble. With the advent of the use of cut-backs we got away from most of the water troubles. It used to be that with nothing but SC oils you had to watch water all the time. We got away from that by going to cut-backs. Where the SC's are still used they generally go to the lean side and sacrifice ultimate durability to get away from water. There is still quite a lot of water trouble which isn't recognized as such, and I think this is a good place to comment on it.

For instance, on an MC mat I have seen cases of MC mats pushing and failing when the failure was blamed on excess oil, when actually it was water. You could always tell it because the MC had not set up in that particular mat. If they would scarify and dig up the mat (they are a little hard to dry out), when it dries out it immediately goes down and is quite satisfactory. I believe lean mats and water trouble are still the principal causes of failure. There are other troubles, but these are the outstanding ones.

Mr. Reagel: Does anyone else have any troubles? What would you expect to happen, Mr. Nevitt, if you had 5 per cent of free water in an aggregate and you attempted to add 5 per cent of SC oil? What would be the ultimate outcome?

H. G. Nevitt: Missouri I think it depends a lot on your conditions. I think this question of water troubles is one that hasn't been studied as much by bituminous engineers as it might have been. We rather ducked the issue by going to the cut-backs, but about ten years ago when we weren't doing anything but SC oil work, practically, I am thinking of the Rocky Mountains at that time, because there is where they were doing most of the work with which I am familiar, we were getting a great deal of water trouble.

The Materials Engineer of Idaho at that time did a great deal of studying of failures, particularly pushing and shoving, and he came to the conclusion that every case he had was due to water, even though in many cases excess oil was blamed. Ordinarily, when you have water and oil together, there is enough colloidal clay available, or in the mixture, to cause emulsification. You immediately get a mixture which looks just like oil, only it tends to be more viscous. You put that down and you have an excess binder. The binder is not excess oil; it is excess emulsion, which doesn't break because it has been made in place and is not a breaking type. As soon as you get warm weather you are almost certain to have corrugating, bleeding, shoving, depending on the percentage of water and oil, so ordinarily I would say in your condition you would have considerable trouble with bleeding, possibly immediately; certainly by the following season, if there were any hot spells.

Occasionally with cut-backs you can mix your wet aggregate and not get emulsification. The solvent, in the drying of the material, if you give it aeration, carries the moisture with it. That was brought out in North and South Carolina practice, described here in the last day or two. I have seen a number of cases of that. That is one reason I say you get fewer water troubles with

cut-backs, but occasionally you will get the emulsification previous to the aeration of the solvent, and either the globules of the asphalt in the water mixture prevent the evaporation of the solvent or, if you have the reverse emulsion the globules of water in the asphalt mixture prevent the evaporation of the water and you do not get your drying action, and you will have later trouble which can be cured, generally, by further aeration, because it will finally dry out.

Mr. Reagel: I would like to check you on that except that we found that the final action, then, is a mat that is apparently too dry. In other words, the greater evaporation of moisture leaves a rich mat on top and a lean one below, and the claim is that we have too lean a mat.

H. G. Nevitt: Missouri I would check you 100% on that. When you get emulsification, the engineer thinks he has a rich mix. Or, if you have a suitable mix, the total volume of emulsified oil and water is such that the mix won't tolerate it and it will travel away, either by bleeding and being carried off by traffic, or it will migrate through the subgrade. It will get away because it is an excess, and when you later lose your moisture you are back to a lean mat, just as you say.

Mr. Reagel: Are there any other questions on that point? I have another question: "What should the solvent content of mixes be at the time of laying the mat?"

I believe Mr. Davis of Texas answered that from the platform yesterday. The statement he made was that they should be entirely free of the solvent if possible, but that the mat could not be laid in that condition so the next best answer was to have as little of the solvent left as possible. Does anyone care to discuss that question further?

H. G. Nevitt: Missouri I don't like to talk too much, but I would like to enter a contrary opinion there. I think you want to get sufficient solvent out to have a mixture which has sufficient stability to carry the immediate traffic. The amount of solvent that you have to get out to do that is dependent, of course, upon the aggregate, the bituminous material used, and all the other factors on the job.

The Nebraska people report that in most of their work 50% of the solvent going out gives them a sufficient amount, and they control their jobs on the basis of the solvent going out. In discussing it with them they maintain that that is a more satisfactory control for their aggregates, which are very much alike, I understand, than a field stability test. I don't know; I am not sufficiently versed in stability tests and the possibility

of making field tests to know whether you could control your stability by a field stability figure and let that be a measure of the solvent removal, or whether you do as Nebraska does, actually test the solvent that is going out; but I don't believe you want any more solvent out at the time of laying down than is necessary to make sure of stability. I don't believe you want all of the possible solvent out, althought I am not saying you should deliberately try to keep too much in. I think our bituminous roads are improved by a kneading action under traffic, when the particles still have enough freedom of movement to compact and give you a better density while the solvent is going out.

I believe that is one of the original reasons that the MC type was developed in addition to the RC, that it gave you a period of setting up in which time your mix was sufficiently stable to not shove or rut under traffic due to too thin a liquid binder, and yet gave you some flexibility and a chance to adjust itself to internal movement and develop a more dense mixture under the action of traffic before your final set-up.

Mr. Reagel: I think perhaps Mr. Davis was thinking of the very common trouble with which most of us are familiar, that of trapping too much volatile in, which results later in a serious alligator cracking on the surface.

W. W. Zass: Arkansas I raised that question yesterday, and I think this particular question today refers to what might be the critical point of contained solvent, particularly in MC or RC mixtures, not so much in reference to SC mixtures.

Now this question arises: We say there is a critical point. Where might that critical point be? Is it eight per cent by weight, or ten per cent or twelve per cent, rather than fifty per cent, as Mr. Nevitt has just spoke of, of the amount of solvent originally contained? The various types carry a different amount of solvent. Some may carry as much as 35 or 40 per cent. Necessarily, then, if you took 50 per cent of each or all you would get a wide range in itself, and it would appear to me that there must be some critical point at which the maximum amount of solvent would be deleterious to the mix.

Mr. Reagel: Certainly we can measure that in terms of time, I think, because some of the states have tried varying periods of aeration. Does anyone have any information as to what that would result in in percentage of solvent?

H. G. Nevitt: Missouri Quite a few years ago we did run quite a few tests on the degree of drying versus the effect on the material. Our theory was based on a viscosity time effect. We weren't concerned with the amount of solvent so much

as the viscosity that you had by the loss of your solvent, because we thought that if you reached a certain viscosity you then achieved a certain stability, and while I don't remember the actual times—in fact, we don't get them in field tests—we were inclined to make a comparative study rather than a field study, because in the field we think you are tied up entirely to your individual job. You get drying, remember, by both aeration and absorption into the aggregate, and that varies greatly from job to job, but we did notice that the more solvent you had in the material the quicker you lost solvent on a percentage basis, so that you tended to come down to very close to the same drying time. For example, if you had twice as much solvent you would not have twice the time to get down to an arbitrary 10% left in the mix, regardless of what you started with. You would get to that 10% not as quickly, but almost as quickly with the greater amount of solvent, because the thinner material dried more rapidly, having more solvent and a higher partial pressure of solvent to be forced off.

I think you have to come down to a stability measurement rather than the percentage of solvent left. As Mr. Reagel points out, you must not have so much solvent left in that when that solvent is lost you get distinct shrinkage cracks. In other words, you must not have your particles so far separated by asphalt and solvent that there is a distinct shrinkage when the solvent goes out. I don't believe that is usually observed when you have normal aeration. The 50% rule for Nebraska I do not think is a general rule. I don't think they intend to apply it to all cases. I simply state that in their practice, which is limited to similar aggregates and not a very wide range of distillate percentages in their materials, they have found it quite satisfactory.

Mr. Reagel: How long did it take them to get the 50% out?

H. G. Nevitt: Missouri I think the 50% comes out very quickly in some jobs, and at considerable periods in others, and they refuse to go to a time element for that reason, depending on the aggregate and the job conditions. They actually have a field test in which they determine the lay-down time for the contractor by the percentage of solvent left in the job.

W. W. Zass: Arkansas I would like to ask Mr. Nevitt if he would recommend a specification requirement as to the amount of solvent that may be contained in a mix at the time of spreading.

H. G. Nevitt: Missouri Personally, if I were recommending any specification for highly general use, one for all types of aggregates,

I would restrict myself to a stability specification. I would set up some sort of stability or compression test. I am not trying to get into an argument as to what stability measurement to use, but some sort of stability measurement with, perhaps, where it was found possible by experience (I don't know whether this would come very often), an arbitrary limit to take care of the shrinkage which will show up in a few cases. I believe a general specification should be primarily a stability specification sufficient to carry the traffic without maintenance difficulties until further setting up had occurred in the road.

Mr. Reagel: Does anyone have a definite recommendation for a percentage or time for aeration? If not, I have another question: "What should be the maximum size of aggregate used in a 2" to 3" mat?" I think that is a good question.

I. E. Russell: Wyoming

On two jobs we had this summer the mat was an inch and a half MC mat, and previous to this year our maximum size was specified at ¾". This past year we went to 1" maximum size, in line with the Bureau recommendations set up last year. On these two particular jobs there was a good percentage of crushing in the pit and we found that there was a good deal of float developed in the laying down process. That is not particularly true in the thicker mats. In other words, where they have more material to work with they can apparently use a 1" maximum size without any trouble, but on the thin mats, the 1½", you do develop float, and it is necessary to get right behind it and seal it before it starts to ravel. You can stop it pretty well all right by doing that, but you have to follow pretty closely with your seal coat on those particular types.

J. W. Powers: Arizona

In my observation the maximum size that can be used in any mixture many times depends upon the ability of the lay-down man to lay it down properly. In laying any thickness of mat, regardless of whether it is 2" or 3", if you attempt to lay it in too thin courses, you can bring all the rock to the top and have it there as float; whereas, if you have an experienced blade man who can keep the rock more uniformly distributed from top to bottom, you can possibly lay 1½" rock in a 3" mat, but certainly not more than a 1" rock in a 2" course, so the size you set depends upon the skill of the operator in getting it into the mix.

I am speaking particularly of road mix jobs and blade lay jobs. That isn't necessarily true when you are laying it with a finishing machine, although there is some dragging action which has a tendency to roughen

up the surface. Somebody described it yesterday as a herring-bone effect.

Mr. Reagel: We have noticed that the closer the aggregate approaches the thickness of the mat, the more tendency there is to this effect, following fanning or light blading.

We find another Proctor question. Apparently on the program yesterday or the day before someone discussed the use of the Proctor method in connection with clay-gravel construction. The question is, "How do you use the Proctor method in the study of compaction in the case of the clay-gravel method, when you have a large metal content in the mixture? Can that be done, and if so, how?"

Does anyone use the Proctor method for determining compaction of a clay-gravel stabilized base?

A. E. Stoddard: Iowa The Iowa Department has been using that. We find no difficulty with it. We provide our field men with a Proctor mold and an arrangement for compaction that we have developed as a guide so that tamper is operated a little more freely than it would be by just raising and dropping it by hand. We feel we get a little more uniform results between different operators by doing that. We don't use the Proctor needle. We use the mold for the determination of the density of the compacted base. We do not believe that the use of sand for determination of the volume is nearly as accurate as the use of a liquid material.

Mr. Reagel: You combine the Proctor compaction with a volume determination of density. Does anyone else have a suggestion?

We have an old friend here: the definition of a seal coat. I think someone has been a little reticent in coming forth with a definition. Can't we have someone offer us a definition of the dividing point between a seal coat and a surface course?

H. G. Nevitt: Missouri I would just like to add a suggestion or put in a suggestion, that we take two names. A seal coat is where you are essentially putting material on top of the old surface and merely adding enough aggregate to prevent picking up, which is negligible from the standpoint of increasing the thickness of the mat; and any other treatment, in which you add an appreciable layer of aggregate and therefore appreciably increase the thickness of the mat, should be given some other name, such as armor coat or protective coat. You may seal with aggregate, but you may not seal at all even if you build another mat on top of it, and the new mat you put on may be no more impervious to moisture than the original mat.

I have seen many cases of adding a mat which didn't have any sealing effect at all, and I wouldn't have called that a seal coat. It may have done the job it was intended to do, which was to prevent failure of the original mat, but I don't think it was a seal coat; so I would suggest, as a start, that we differentiate between seal coats, which are essentially a layer of bituminous material intended to soak into the old mat and cover the surface and add very little to the thickness, and other types of coat which build up the thickness.

Mr. Reagel: That is closely akin to the idea that a seal coat is intended to seal, and a surface course is intended to provide some additional wear. I think a seal is a seal by any other name. Perhaps it might be desirable for us to start some procedure to call the same things the same name. I have noticed throughout this meeting that some of us were talking about subgrades and bases and surface courses all at the same time, and perhaps the names were considerably confused. I think it would help us if some organization or someone would get a terminology or a set of names that would get us all talking about the same thing.

J. W. Powers: Arizona There is one point in that connection that might be of some particular value, and that is the fact that the Bureau of Public Roads, at least in our particular district, will not pay for what we call a seal coat, except on newly built surfaces, which consists of an application of a flush coat of whatever type of material we want to use, whether it be MC, SC, or emulsion, and over which is usually placed from 20 to 25 pounds of screenings between 3/8" and No. 10. They consider such an application on an old road, maintenance. They will participate, however, in construction of a very similar nature which carries a minimum of 35 pounds to the square yard; thus differentiation between a seal coat and resurfacing might well be taken from the Bureau's interpretation of maintenance. They consider that anything from 35 pounds of screenings to the square yard on up, with the necessary amounts of oil, as being some form of resurfacing or retreatment.

Mr. Reagel: I have a rather similar question here, using the No. 10 mesh sieve as an example: "It is noted both in conversations and papers that there is a promiscuous use of such terms as No. 40 sieve, No. 40 mesh sieve, etc., and the question is asked, are they of the same size and, if not, what is the proper designation of each and what authority is set up for proper designation of sizes from No. 4 down?"

I. E. Russell: Wyoming That is something on which you can easily become confused, but in our specifications we state the sieve and

behind that we give the numerical opening. That definitely sets up the size so that everyone knows what you are talking about.

A. E. Stoddard: Iowa — I believe the A.S.T.M. has a schedule of sizes which are definite, if anybody wants to refer to them.

There is no reason why that could not be the correct designation to use. I think that is common practice with other concrete men and everybody else, to refer to the same schedule as the A.S.T.M.

Mr. Reagel: I think the information is available to us all right in that way. I think perhaps the question was directed at the tendency in papers to talk rather freely from one to the other without setting forth just what the size opening might be.

A third question in this series: "Does vegetation along the roadway actually have any beneficial effect in lowering subgrade moisture?" Does anyone have any definite ideas on that?

That was suggested by California, in a paper yesterday or the day before, as a possibility. Mr. Stokstad, did you have anything to offer in that connection?

O. L. Stokstad: Michigan — What I have to say on that subject refers only to the climatic environment characteristic of northern sections. Roadside vegetation probably does not assist very much in drying out the subgrade. Rather is may have the opposite effect. Vegetation, as it grows in ditches and on shoulders may sufficiently slow up surface run-off to actually increase the moisture content of the subgrade.

Vegetation may be used as an indicator of subgrade moisture conditions. A luxuriant growth of water-loving plants, for instance, indicates to us the necessity of checking closely for detrimental subgrade conditions.

W. H. Wood: Texas — We have in our state, of course, several types of climate. In the western part of the state we have a semi-arid climate; I wouldn't call it arid, but rather dry. It has been the practice in that end of the state not to remove any vegetation from the shoulders because it helps to hold the soil. We have observed the fact that in the semi-arid areas the vegetation does grow along the side of the concrete roads. I don't think there is any possibility of vegetation taking all the moisture out from under such roads, because the moisture continues to rise from below and still the road is perfectly dry.

In the central and eastern parts of the state we quite often remove the vegetation for the purpose just expressed here, to keep the water from accumulating on the shoulders and staying too long.

In addition to that, some consideration should be given to the fact that if you left vegetation, particularly the short grass of the type we have, called Bermuda, grow up against the side of the job, you might find that you were neglecting your maintenance and the shoulder might be quite low without your realizing it. We have one case in mind where a concrete slab cracked up after the Army had run a bunch of trucks over it, and we found in that case that the shoulder had shrunk down below the bottom of the slab and about three or four feet of the slab projected into the air, with no contact at all. That could have been noticed and would have been noticed and fixed up had the vegetation been kept off the shoulder.

O. L. Stokstad: Michigan — I didn't mean to infer by my prevoius statements that we try to destroy vegetation growing in the ditches in order to speed up run-off. We encourage vegetation in the ditches and on some roads we encourage vegetation on the shoulders, because of its aid in protecting the highway investment from destructive erosion.

W. H. Wood: Texas — We keep vegetation in the ditches as much as we can. We just take it off the top of the shoulder.

Mr. Reagel: Is there any further discussion in that connection? Those are all of the questions that have been handed to me from the committees. Does anyone have any subject that he desires to discuss, or is there anyone you desire to put on the spot here?

J. G. Schaub: Michigan — In resurfacing old worn out concrete roads with bituminous mats there has been quite a bit of discussion about the thickness of the mat and the tendency for the cracks in the concrete to appear through such bituminous mats. I would like to know if any state has had any experience with the idea of cracking up the old concrete pavement to, in effect, destroy the expansion and contraction, and then applying a wedge course and a bituminous mat. Has anybody had any experience in trying that out?

J. W. Powers: Arizona — We have done exactly that thing. In order to make a comparison between what we could expect on a badly cracked concrete pavement which was being resurfaced with three inches of cut-back asphalt, plant mix and machine lay, we took a strip, as I remember it, 1,000 feet long, and broke it up with a heavy ball on the end of a drag line, and that is the only piece of pavement with this 3 inch cut-back mix that does not have any cracks. There was no attempt made to particularly smooth up the surface other than to place a small amount of plant-mixed material on top of it so as to give the laying machine a fairly uniform base on which

to ride, but so far as breaking it up, it was not very thoroughly broken. It did not displace particularly, so the amount of material that was put back to pad it out was not material. As I said, that is the only place that did not crack and check.

Mr. Reagel: What do you say, Mr. Powers, as to the approximate or average size of the pieces left in that pavement?

J. W. Powers: Arizona — It would be my judgment that the pieces were probably somewhere in the neighborhood of 6x6 up to possibly 12x12.

O. L. Stokstad: Michigan — I would like to ask Mr. Powers what kind of subgrade the pavement was resting on.

J. W. Powers: Arizona — The subgrade on which it was resting was a sand-clay-gravel type in which the binder was principally A-7 type. There probably was in the neighborhood of from 15% to as much as 30% minus 200 of an A-7 type. The concrete pavement had cracks varying from comparatively minute cracks to as much as an inch and a half wide.

J. G. Schaub: Michigan — Did you attempt first to go over this with a heavy roller after having cracked it, to set the broken pieces?

J. W. Powers: Arizona — Not as I remember.

Mr. Reagel: I might say in this connection that we have had a similar experience with the failure of rather thin layers to cover or refrain from cracking. They have reflected all the cracks in the base. To get away from that effect in some later resurfacing we went to about a 5½" binder course plus surfacing, and found that we got away from the cracking effect. I am balancing some such procedure as that against the expense of breaking up the concrete.

J. G. Schaub: Michigan — Along that line I might cite an experiment we had in Michigan in connection with a broken up pavement. This was an old concrete road that had been laid over a swamp section paralleling a railroad, and the swamp was giving us a great deal of trouble. Last fall, rather suddenly, due to some unusually heavy hauling that appeared at that time of the year, the old concrete pavement broke into very much smaller pieces than it had before and some of the pieces started turning up. We had to close the road to traffic, because it was dangerous. In the emergency something had to be done, as winter was approaching, and we applied a bituminous mat. The suggestion was made by our Bureau of Roads representative, Mr. L. P. Scott, that we might try reinforcing so we placed the ordinary expanded metal reinforcing mats on the concrete pavement after wedging and chink-

ing the cracks.. After wedging up with a hot mix binder to get a level surface, we then placed about four inches of binder and an inch of bituminous concrete top on it, and so far it has effectively stopped any movement underneath. How long it will do so, of course, nobody knows.

We figure that any thickness much less than that might crack up with some slight movement of that steel underneath.

Mr. Reagel: Gentlemen, this is your session. We will stay here so long as you keep it alive.

J. H. Swanberg: Minnesota

Has there been any work done on stabilizing gravel aggregate with bituminous material, and possibly using a concrete paver, stabilizing the aggregate with bituminous material such as emulsions or cut-backs, instead of stabilizing with clay? In other words, where we would normally lay a clay-stabilized base, have we considered the possibility of using bituminous material instead of clay?

We are also interested, of course, in how the thickness of that should compare with a normal stabilized gravel base. We are interested in any information along that line.

W. J. Slavin: Michigan

I might say that during the past summer we have done some work along that line. The purpose was to widen the old existing gravel base to a sufficient width for a bituminous mat, and what we did was to add about three inches of new gravel six feet wide along each side of the old narrow base, then scarify to a depth of about six inches. As a rule, the scarifying went into the subgrade to some extent, and if it happened to be a clay subgrade we added a little more gravel so as not to have excessive clay in the mix. In case the subgrade was of lighter soil, gravel or sandy, we added about 10% of clay. Then we applied a gallon to the square yard of SC-3 (on some jobs SC-1 and on others SC-3) and mixed that in, adding a sufficient quantity of water to facilitate mixing, and rolled it in about one-inch layers and finally capped the old base with 2″ of the same mixture. The mixing was done with a specially built multiple blade mixer six feet wide drawn by a "Cat. 60."

T. R. Perry: Iowa

Answering Mr. Swanberg's question, I would say that this week we are starting such a project, using pit run gravel, which is a rather dense-graded material. This aggregate will be plant mixed with emulsion and compacted to a five-inch thickness. A surface treatment using emulsion will be placed on top of this base. Maybe a year from now I can tell you what to expect from this type of base.

J. H. Swanberg: Minnesota — What equipment are you using to mix that with?

T. R. Perry: Iowa — A 27-E concrete mixer. The contractor started out to use a Barber-Greene traveling plant, set the Barber-Greene up as a batch mixer, but it wouldn't work out, so they switched over to the concrete mixer.

J. H. Swanberg: Minnesota — How much emulsion are you using?

T. R. Perry: Iowa — About 4% asphalt content and the asphalt in the emulsion is running 60 to 62 or 63%.

R. N. Kinnaird, Jr.: Mississippi — On a WPA project we were employed to design and control a bituminous paving using aggregate in stock which had been purchased by the Procurement Office of the WPA and mixing this material in an E-27 mixer. The materials used were a pit run sand and gravel, mineral filler and RC-1. The mineral filler consisted of crushed limestone with approximately 90% passing the No. 200 sieve.

The aggregate was dried before placing in the mixer in an improvised oil burning drier. The batch consisted of 1790 lbs. of mixed aggregate, 80 lbs. of mineral filler and 130 lbs. of RC-1. This was mixed for 5 minutes, dumped and stock-piled on the job for about a week. The material was then placed on a prepared bed (generally an old clay gravel street) from 2 to 4 inches thick, about 4 inches in the center and 2 inches on the outside edge. This was rolled and traffic withheld for a week or 10 days. At the end of that time the surface was then sufficiently dry to support the light weight traffic these streets obtained.

During the first three months only the top 2 inches had lost the volatile constituents of the RC-1 and the base still contained a large portion of the naphtha. Density tests made on the pavement seldom showed a density in excess of 80% of the theoretical and as a whole stayed between 75% and 80%. This pavement has been down approximately one year and has given satisfactory service as a residential street; although it would not be satisfactory on more heavily traveled streets.

Though from an engineering standpoint this pavement is not satisfactory, the use of it as a dust palliative is good, so good that the town officials proceeded to put down more pavement of this type on a second project.

A. E. Stoddard: Iowa — Was that mixed with a 27-E?

R. N. Kinnaird, Jr.: Mississippi

A 27-E with a one-ton batch. It may have been more than that. We may have put a yard in there. I don't recall.

Mr. Reagel:

The question was asked me today, in connection with a committee that you had here last year on the desirability of having a bibliography on bituminous subjects. At that time you were informed that the American Association of State Highway Officials were working on such a project in cooperation with the Highway Research Board, and I was asked what the status of that might be at the present time.

I would say, in answer to that question, that the work is well under way. The material has been assembled and the printing will proceed possibly or probably during January or February, and the completed material will be available very soon after that. The information to be printed will be something more than a bibliography in that there will be a sentence or two of description of the scope of the various projects covered. I might say that the subject matter is a bibliography of research projects, current and completed, of highway departments, major schools of the country, and to some degree commercial concerns. It covers the entire highway research field, however, not just the bituminous field, and it is hoped that the issuance of this bibliography will form a nucleus for cooperation with various others that haven't been contacted at present.

If anyone desires any more detailed information in that connection, Mr. Crum of the Research Board is here and he will be glad to go into the matter further with you if you desire.

Does anyone have another question on any subject he would like to have discussed?

J. G. Schaub: Michigan

I would like to ask a question as to what experience some of the states in the northern section have had on applying surface treatments by the distributor and chip application method to gravel surfaces, with MC and RC cut-backs and the relative merits of the two types. I presume, of course, in every case they would be using a prime of MC. What is the relative experience with the MC materials against RC?

Mr. Reagel:

Mr. Perry, would you care to answer that?

T. R. Perry: Iowa

We have used RC-4 in two instances and the majority of the other surface treatments have been with MC-4 and MC-5. We have also used some of the heavier SC's. On the RC-4 we are not all in accord as to what is the matter, but we have had rather serious failures. I think that our climate is a little too severe for the

material that we used. Possibly if we had used a softer material we might have had better success.

J. G. Schaub: Michigan — Do you use a prime?

T. R. Perry: Iowa — Yes. On both cases a tar prime was applied beneath a double surface treatment of .3 gallon of RC-4 and 30 pounds of aggregate per square yard.

J. G. Schaub: Michigan — The thing that prompted that question was the thought that on highly stabilized gravel bases, if you use an MC prime you don't get much penetration. The tendency would be for some of the prime to remain on top, which would overcome somewhat the hardness of RC material by softening it with this excess prime that does not penetrate. I wonder if that will overcome the difficulty where the harder RC material used alone causes some ravelling.

T. R. Perry: Iowa — In these two cases it was not a stable base, you might say—not of the heavier type. Rather than to depend on mixing the prime with the top bitumen I think we probably would cut down on the amount of prime used.

Mr. Reagel: Is everybody satisfied?

J. H. Swanberg: Minnesota — In that connection I think I covered this the other day, but on our stabilized gravels, where we have treated them the same year, we have had a requirement of a three-weeks interval between the completion of the stabilized gravel base and the mat that went on top or the surface treatment. However, on one job it was necessary to open that road within three days following the completion of the stabilized gravel base. We put the prime on it within three days. We haven't had any difficulty with the penetration of MC-1 or the TC-1 on those stabilized gravel bases. We have found that in the center possibly sixteen feet of the road we could get as much as .2 gallon to penetrate into that base and leave a fairly dry surface, whereas on the outer four feet on either side we could get as much as .3 gallon. We have used, on one job just completed, an MC-1 prime and a double seal of RC-2. We also built one job last year where we used an MC-1 prime and obtained satisfactory penetration (at least we assumed it was) of about a quarter inch, and treated that with a heavy grade of cold application tar.

As I said the day before yesterday, one section where the MC-1 prime had remained as long as nine days before we surface treated it, and the surface treatment was only a single surface treatment with sand cover, was in very good condition. Where our prime was younger we had some pot-holing of that surface treatment.

A. E. Stoddard: Iowa

I would like to ask Mr. Swanberg what method of compaction he used on that base that he got such differences between the edges and the center.

J. H. Swanberg: Minnesota

Most of our stabilized gravels are compacted with these pneumatic rollers. I think the answer to Mr. Stoddard's question is that those roads are open to traffic all the time during construction, and then our gravel is hauled for our bituminous surface courses over those roads during the placing of the bituminous mat, and there is a tendency for the traffic to concentrate on the center portion of the road. I think that is the answer to the difference in compaction.

W. H. Wood: Texas

I would like to ask Mr. Swanberg: "Can't you divert your traffic at all? May I ask if he could not divert his traffic from the center of the road by placing brush? We found that to be very effective. It was somewhat annoying, but small trees and brush that would not hurt a car if it ran over them can be placed down the center, and the traffic will stay off the center. That makes it compact the outside edges.

J. H. Swanberg: Minnesota

No, we did not do that.

Prof. W. S. Housel: Michigan

There was one question discussed earlier that I was was interested in and might have a helpful suggestion to make, and that was the question of how much fluid material or how much solvent there was in a mixture at the time when it should be laid down. Mr. Nevitt stated that the stability would be the criterion, but that of course requires some sort of stability test. I am just wondering if it would not be possible, in that connection, to use another control method that would be easier and more rapid. We know, of course, that the maximum stability is definitely related to density, and density in turn is definitely related to the percentage of voids you have in a mixture at any time. How much fluid material the mixture will accommodate without loss in density is going to depend on the grading, so you can't specify any particular limits of fluid material without specifying whether it was a dense-graded mixture with a small percentage of voids or a poorly graded mixture with a large percentage of voids.

I wonder if it would not be possible, in terms of the thickness of the material spread down as related to the voids, as compared with the thickness of the mat at maximum density at the time it was compacted, to set up some sort of formula.

H. G. Nevitt: Missouri

When you say "density" are you referring to a compacted sample? You appreciate that while the material is being aerated it is in a windrow being moved back

and forth and is not compacted. Are you suggesting that we have a standard compaction and then take the density?

Prof. W. S. Housel: Michigan

I would require a standard compaction test.

H. G. Nevitt: Missouri

The difficulty there, Professor Housel, is this: The binder with the solvent still in it, first of all is giving you a greater film thickness, which is increasing your lubrication effect, and also has a lower viscosity. As a result you quite probably, or possibly, would get a better density before you had had aeration than afterward, because you had better lubrication and therefore better packing. In other works, any field compaction test might end up with a higher density on your unaerated mixture than after aeration.

Prof. W. S. Housel: Michigan

In any case, then, your total amount of fluid binder in there is always going to be less than the percentage of voids in the compacted mixture.

H. G. Nevitt: Missouri

You were dealing with a light material first of all, and the effect of the voids by the change in aeration is going to be small, and the effect on the weight is going to be small from the standpoint of laboratory tolerance. You are not going to change a great deal in the weight per ton by the evaporation of your volatiles, and while there might be some theoretical tie-up between the two, from the practical standpoint I would be concerned primarily with laboratory error. Your tolerances on your various procedures might be greater than your correlation between your dense and less dense mixtures due to the amount of volatile present.

Prof. W. S. Housel: Michigan

By "laboratory error" do you mean variation in aeration?

H. G. Nevitt: Missouri

I mean in compaction; in other words, in forming your mixture, not in weighing it—in forming the compacted mixture under field test conditions.

I would like to make a remark or two on this subject of aeration, since we have it up. You know, we didn't use to aerate mixtures at all, even in the early days of cut-backs. A few places here and there did, but I am talking about it in a general way. We did begin to have troubles in certain areas, but outside of those areas they weren't having so much trouble.

Well, in these places they did have trouble, and the Nebraska people are the outstanding examples of trouble without aeration. They had to go to aeration,

and they badly needed it, due to the type of gravel, mostly Platte River sand, a rather smooth, small-sized particle material. We got into aeration, and it suddenly became very fashionable to aerate, but I believe there are still lots of places in the country where they are aerating where they don't actually have to. Possibly here and there they will get a little rutting, but I am not so sure that the ruts wouldn't iron themselves out. However, we hesitate to oppose aeration because it is unquestionably a precautionary measure with nothing against it except expense.

That is, if you do aerate, first of all, you will not get rutting on your job. Furthermore, you help remove the moisture, and in certain types of mixes, of which the sand mixes are very characteristic, you just don't dare put them down without getting the moisture out, so aeration is fine for them. But I have been in parts of the country where, just as a matter of personal opinion, I thought they were paying a lot more for aeration than this insurance plus the actual benefit where they needed to do it was worth. I think aeration is possibly being overdone in certain areas.

Just stop and think! We used to use these thin SC materials, corresponding to SC-2 today, and you couldn't aerate them because they don't particularly change, and we got along fine ten or twelve years ago. They didn't have any aeration because they didn't have any materials in most places that needed aeration, but they laid them down and they got along in good shape.

It is true that you get better jobs in lots of cases by aeration, so I am not opposed to aeration at all, but I do bring up the discussion and raise the point that where aeration is beginning to get expensive, is causing the contractor a lot of trouble and running the bids up on jobs, it should be considered rather carefully and a more common sense attitude taken toward the possibility of not needing so much aeration or worrying too much about the percentage of volatile left.

I am not certain that there is a great deal of failure or trouble due to excess volatile left in the mix, in the general type of mix, although in certain types, particularly in the fine materials, you do have to watch that particular point. The outstanding examples, of course are these fine sand mixes, but there again I believe a lot of aeration troubles can be cured by going to a type of bituminous binder which is easier to aerate and less expensive.

W. W. Zass: Arkansas

Would your remarks apply specifically to MC or RC products in relation to dense mixtures with possibly 5 to 10% passing 200?

H. G. Nevitt: Missouri

I was making a very general statement. I think it is more true in MC's. You very rarely go to great expense in aerating RC's. By the time you have thoroughly mixed your mat you probably have sufficient aeration and you probably benefit enough from a better, more uniform mix to that extent. But I have heard of jobs where they have made as many as a hundred turns on the blade to get aeration, and it just sounds to me like a little too much trouble for that. Of course that is unusual, but I know that in many cases aeration is excessive, and I don't believe, in your ordinary dense-graded mix with a fair percentage of coarse material, showing some stability, that aeration has to be overdone. It must be watched. It comes to the point made by Dr. Winterkorn a couple of days ago. We are supposed to have individual judgment and not too many rules instead of judgment. Unquestionably there are lots of places where you should aerate and it should always be watched, and I am merely making a plea for not overdoing it to the point that your jobs are unduly run up in expense without the benefit really being assured from that expense.

George Fry: Arkansas

I want to ask Mr. Nevitt if he thinks the fact that the fact that the Platte River material in Nebraska not being crushed has any effect.

H. G. Nevitt: Missouri

Yes, I think it has a considerable effect. They have jobs that you can't put down without aeration. I believe in the stability test, and when I say that I mean a rather rough and ready stability test. All you have to do is get above a certain limit. Just make your test rough and ready and set your limit high enough that you are safe. Don't go to a very elaborate test. Go to some rough and ready method, but be sure you are above that limit and I don't think you will have to worry. I think in lots of cases the job will be above that limit by the time the mixture is thoroughly coated and the aggregate has been turned over sufficiently to get proper mixing for other reasons.

Mr. Reagel:

By way of explanation, I might add that in connection with the rather stringent aeration requirements of Nebraska they have to deal with considerable blow sand there.

H. G. Nevitt: Missouri

I am not criticizing Nebraska's practice at all, because that is one place where I know they need it. I am just talking about the extension of that practice more or less by hearsay to areas where they have had occasional trouble and needed aeration on such jobs, but have adopted the idea that when you make a cut-back mixture or any kind of mixture that will aerate you have to aerate. I don't believe that is true.

J. H. Swanberg: Minnesota

In connection with the remarks of Mr. Zass and Mr. Nevitt, I might say that in Minnesota our gravel will run approximately 2 to 4% passing a 200-mesh sieve, and our mixes from 4 to 6%, possibly 5% of MC-2. Immediately following the mixing of those mats, or the next morning, we lay the mixes, and we have had very little trouble from shoving because of the greenness of such mixtures. Of course our mix is, you might say, open. We have only 2 to 4% passing the 200-mesh sieve, in some places as high as 5%.

J. W. Powers: Arizona

In Arizona we use MC-3, MC-4, and MC-5 and for road mixing MC-3 or MC-4, and by the time we are through road mixing the aeration is taken care of to the extent that you don't have any trouble. With the MC-5 we use it in plant mix and lay with either a finishing machine or blade. We lay it down without any aeration at all. Our maximum limit on minus 200 is less than 5%, and most of the time it will run to probably 2 or 3%, with generally between 3.6 and 4.2% asphalt. The aeration doesn't mean a thing. The relative humidity in the day-time doesn't run over probably 10%.

W. W. Zass: Arkansas

Generally, what per cent solvent would you say was in your MC mixture after you had laid it?

J. W. Powers: Arizona

I would judge it was somewhere in the neighborhood of 10% on the road mix. That is purely a guess on my part, but that would be my judgment.

I. E. Russell: Wyoming

Our experience bears out both what Mr. Powers and what Mr. Nevitt have said, and I think as a rule where you find the grading of the material mixed with an MC is such that you get what you call a fast mix behind the mixing machine you don't need to worry about aeration. I think that is the same idea Nevitt has.

H. G. Nevitt: Missouri

I might make a little comment here. There is another angle that we haven't brought up that might be of interest and might have some bearing. The drying of cutbacks, particularly MC's, is a double function, as I commented previously. You have both loss of volatile in the air and you have absorption of volatile in the aggregate. It is possible that in certain types of aggregate it might be a good idea to get aeration for fear you had too much absorption into the rock and it stayed there; in other words, if you had a sufficient preliminary drying to give you your stability and could lay it down but you could never get a further drying. That is purely speculation. Possibly there have been jobs where the aeration gave them better results, yet the stability was apparently good. In other words, you had more of a curing of the mixture, and I just want to bring this up to take care of the eternal exception to any comments

you make that pops up and infers that you must have been wrong.

There may be occasionally types of aggregates or types of jobs, where aeration gives a better film, due probably to this situation I have mentioned, so if you have that by experience, of course be guided by that experience, The thing I am trying to emphasize here is, as Mr. Powers and Mr. Russell and others have brought out, you can often get along without aeration, and when you can, let's not pay for it. But when you find by good practical experience and trial that you need it, then go to it to the extent that that experience indicates it is worth while.

George Fry: Arkansas — I would like to ask Mr. Powers and Mr. Swanberg about what is the maximum size of aggregate they use in these road mixes, and approximately how much of that material is crushed. Of course you will have different pits; I understand that. But just approximately?

J. W. Powers: Arizona — In our case it happens to be that we have a specification which calls for a minimum of 40% and a maximum of 55% retained on the No. 3 sieve. That is regardless of the particular pit we are in. We require the contractor to crush sufficient oversize to give us that much rock or waste sand, accordingly.

Most pit sites in Arizona are gravel deposits. These gravel deposits carry gravel in many cases as large as 24 inches in diameter. Deposits will usually grade all the way down to the very, very fine sizes. Of course, the contractor is interested in the production of the proper grading and in not crushing any larger sizes than is necessary, so it is very, very seldom that he crushes anything larger than about 10 inches in any pit that he may go into, and where the proportion of sand to gravel may be fairly well within the specifications he will only crush enough rock in that particular pit so as not to have to waste sand. In case the proportion between sand and gravel in a pit is such that he is going to have to waste sand anyway, then he will probably crush absolutely everything that he can get hold of, so that we don't, in our case, adapt any pit to the specifications. We adapt the specifications to the pit and work out the proportions so that we get the grading that we want.

George Fry: Arkansas — Your maximum size of aggregate is what?

J. W. Powers: Arizona — One inch square.

J. H. Swanberg: Minnesota

I might say that our maximum size is 5/8" square. Our specifications for road mix require from 25% to 60% passing No. 10; 10% to 35% passing No. 40; and zero to 7% passing 200.

Relative to the percentage of crushing, on our road mixes we don't specify any crushing except that the contractor must crush everything up to eight inches in size. The amount of crushing will vary from perhaps zero per cent up to 25%, 30%, or 40% on the No. 4 sieve. In our plant mixes we tighten down a little on the grading and specify 35% to 45% passing the No. 10 and a minimum of 30% or 35% crushed material retained on a No. 4 sieve.

H. G. Nevitt: Missouri

I just happened to think of another factor which comes into this aeration which might be troublesome to one or two people. I believe that it is a point in which cut-backs, or sources of cut-backs or bituminous materials, vary more than perhaps anything else, superficially. I believe you will find your aeration requirements are tied up a very great deal with the source of the material you use. I may be wrong in that, but it may come in, and states supplied from certain areas will need possibly considerably more aeration than others.

W. H. Wood: Texas

I believe Mr. Zass is more interested in the RC type, aren't you?

W. W. Zass: Arkansas

RC and MC, both.

W. H. Wood: Texas

We have put down quite a little of the RC type and we find that you need no laboratories whatever to tell when it is ready to lay. The inspector determines that. If he airs one out too far, that is the last time he will do that. We can tell when it is ready to lay down, the theory being that you can increase the viscosity of the cut-back to such an extent that the mix can be rolled. If that has too much, you can cut the volatiles out to whatever extent you want.

In connection with the extent of volatiles that may be left in, that would vary considerably, I think. It would vary quite a lot with the type of volatiles you happen to have in that particular cut-back shipment, and also will vary from time to time as you are using that mix. The commercial producers ship quite a lot of that type of material and we make it up. We permit the contractor to mix it on the job or buy it from a commercial producer and ship it to the job. We use RC-2 in connection with that work almost exclusively, and we find that in some instances RC-2 cut-back with the grading we specify makes an ideal mix, whereas with

other stone deposits, notably limestone deposits, the absorption of the aggregate and the percentage of dust which is always present is sufficient to dehydrate the mix, if you want to call it that, and when you start to unload at the other end it is rather hard to stick a pick into the mix.

We offset that by adding a gasoline or kerosene or distillate to it. Kerosene we feel to be the best. We add as much as one pint for 500 pounds, which is a half gallon per ton. That same mix, incidentally, could have been laid at the time it was dropped out of the pug and it would handle all right, but after staying in a car for four or five days and then being shipped around it will lose quite a lot of its mobility, due to absorption.

We have also one gravel plant manufacturing under the same specifications, using flint pebbles which are crushed, so that 85% of the particles show at least one face, and we use silica sand for the fine sizes. That is an open mix, 5% to 15% passing the 80-mesh. The material manufactured at that point is shipped out and is usually on the other extreme. That is, it is too wet. The batch is wet and you have to roll it around, and you may roll it all day before it gets sticky or tacky enough to want to roll out flat on the road.

We have found that shifting to the next grade—in that case RC-3—makes the batch just as hard to unload at the receiving end, so we got the producer of the asphalt to increase a fraction off the 437, which is the beginning of the series of the distillation tests, and bring that up to about 18%, and it makes a fast drying cut-back. That same 18% cut-back, or a fraction off, applied to limestone would be rather fatal, and you might have to use two gallons of kerosene on unloading.

I think a point should be borne in mind that the contractor has the privilege of ordering from the producer an asphaltic material which will fit his particular aggregate, so long as that asphaltic material stays within your specifications. We have found that most producers furnish that cut-back which will work best with their product.

J. W. Powers: Arizona

In line with the remarks that were made by Mr. Wood, I might state that in our state we do exactly the same thing. We ship lots of it out of Phoenix to all parts of the state. We have shipped MC-4, MC-2, and MC-3, and after it has been through the plant and put on the railroad car, shipped and unloaded, it can immediately be laid down without any aeration. The usual type of material that we use for maintenance is mixed from either

MC-2 or MC-3 and shipped into the northern areas. We allow producers in Phoenix to take MC-4 or MC-5 and cut it with kerosene, and make MC-2 or MC-3, and the maintenance men have no trouble in taking it out of the cars and using it for patching material on the roads in any portion of the state. They do not aerate, yet they do not have any trouble with their patches.

Mr. Reagel: Are there any further questions? I have a question which I would like to raise. I would like to inquire if anyone has had any experience in the case of a chert gravel which stripped badly, treating that with hydrated lime, or chemically, and using it as a black base or surface instead of avoiding that material and getting some other. Have you had any experience with such protective measures, Apparently not.

H. G. Nevitt: Missouri I know of a case quite a few years ago where that came up, but there the only change was to change the bituminous materials. Bituminous materials vary in their stripping action, and in this particular case the difference was enough to bring it over the hump and make a satisfactory material. However, there is always a possibility with a stripping product that you will still get into trouble. In this particular case I don't think they used much of it. They meanwhile found another pit and, on everybody's recommendation, changed to that. But they could solve the problem by merely changing the bituminous material.

J. H. Swanberg: Minnesota There is a manufacturer in the Twin Cities using a silica sand and asphalt cement as a final coating material. It is a high type proprietary mix and he is using I believe either one-half per cent or one per cent of a coal tar. I remember a few years ago that Mr. Havey, in the coating of gravels with emulsions, where they were unable to get a coating of certain gravels with that emulsion material, was using some agent other than emulsion to coat the gravel aggregate.

Mr. Reagel: They added some tar in some cases to the mix.

F. F. Havey: Indiana We experimented with all kinds of tar bases, and with crude tar, separated it into very close cuts, and we combined that material with the sulphonated fatty acids so that it could be emulsified with water, so you could get a thin treatment by making a coarse emulsion and spraying it out. You can very markedly change the coating properties of things that are unsuitable on account of stripping or, in fact, not only stripping but that won't coat at all. The film cracks and rolls in beads. It isn't merely stripping. Sometimes you don't have any coating there to strip. I never could tell exactly what fraction of tar was most effective. They all had some effect.

I presume if you wanted to, you could use simply a small percentage of straight tar.

Mr. Reagel: Is there any further discussion of any questions on any subject? Does everybody know everything he wants?

If there are no questions, certainly I think we have been here longer than we expected to when we met for this discussion. It has been a very nice discussion and I thank you for your assistance, attention and patience.

We are now adjourned.

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